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The Farmville High Bridge across the Appomattox Valley

By F. P. Turner*

One of the longest and highest bridges on the Norfolk & Western Ry., known as Farmville High Bridge, has just been rebuilt as a double-track structure in connection with the extension of double-track between Roanoke and Norfolk, Va.

Historical

The line here crosses the Appomattox River and Valley, 145 miles west of Norfolk, Va., and the first bridge was built in the year 1853 as a part of the South Side R.R., connecting Petersburg and Lynchburg, Va. Information obtained from the files of the Engineering Department shows that the road was located after a careful study of the intervening country by C. O. Sanford, Chief Engineer, and built under his direction to secure not only a low first cost but an economical operation.

Briefly described, this line, after rising out of the James River Valley at Lynchburg, follows the top of the ridge which divides the waters of the Appomattox and Roanoke Rivers, in an easterly direction to Petersburg. On this location, grades of 16 ft. per mile could easily be obtained. The one exception to this is near Farmville, where it left the ridge route and crosses high above the Appomattox River Valley, returning again to the summit of the ridge several miles to the eastward.

The Chief Engineer's report shows that several conditions influenced the doption of this route. Chief among them may be mentioned:

(1) The loss of a large amount of tonnage from Farmville if the ridge route had been followed.

(2) The residents of Farmville made an urgent appeal for the line to pass through their town, and raised a subscription to purchase stock amounting to $100,000. This, in the early stages of railroad development, was a matter of considerable moment.

(3) The adoption of the Farmville route would advance the completion of the road one year.

(4) This route was 5 miles shorter than the ridge route.

The greatest obstacle to the construction of this line was the expensive bridge over the Appomattox River and Valley.

First Bridge—The bridge as originally built was 3400 ft. long, varying in height from 60 ft. at the abutments to 100 ft. at the river; clear spans 105 ft. in length. Previous to this time longer bridges, not so high, and higher bridges, not so long, had been built, but taking the length and height together this was thought to be the largest bridge in the world.

From a standpoint of economy, wood instead of iron was chosen for the superstructure, and was also given consid-

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Fig. 1. Placing a Completely Riveted 72-Ft. Plate-Girder Span on the Farmville High Bridge, N. & W. Ry.

eration for the piers. After an examination of a number of higher structures built of brick in several cities of the United States, and having greater unit pressures at the bottom, the successful results obtained warranted the selection of this type of construction for the piers. This decision too was influenced somewhat by the presence in the immediate vicinity of the bridge of a brick clay of superior quality, which would make the estimated cost of the piers about one-third that of stone construction.

tions of those adjacent to the river probably extend to solid rock.

Wood superstructure was anchored to the foundation by iron rods passing entirely through the piers in shafts left for the purpose. The estimated cost of the wooden trusses of the superstructure was $18 per lin.ft., and the total cost of the bridge was $167,500. It was built to carry engines weighing 32 tons, although the ones in use at that time weighed only 16 tons.

Second Bridge—The superstructure was rebuilt in

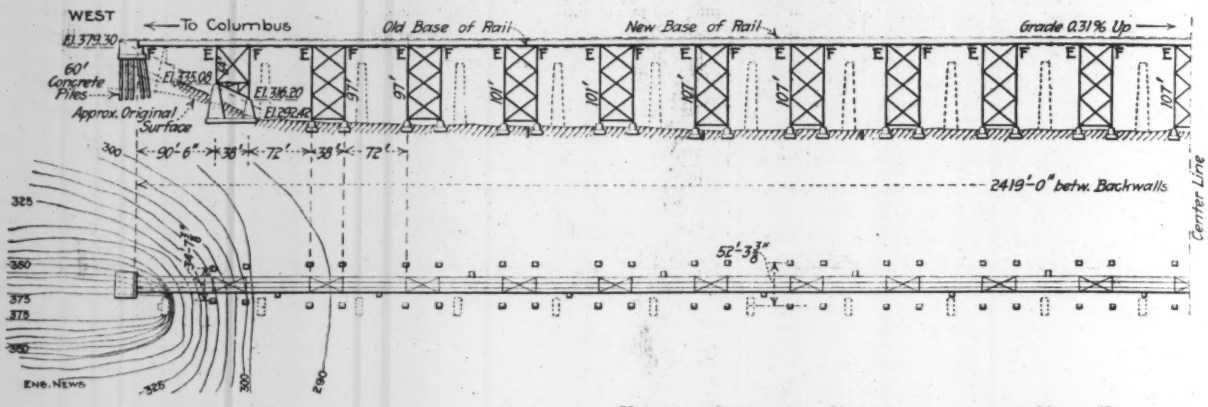


Fig. 2. Plan and Elevation of the New Farmville

The abutments were built of rubble masonry, and the bases of the piers to a point 3 ft. above the high-water mark were built of stone masonry with cut beds and joints. The foundations extended about 8 ft. below the ground, resting on clay and gravel. The piers were of brick masonry 8x22 ft. at the top; 14x27 ft. at the bottom and were 82 ft. high above ground. The foundations of those adjacent to the river probably extend to solid rock.

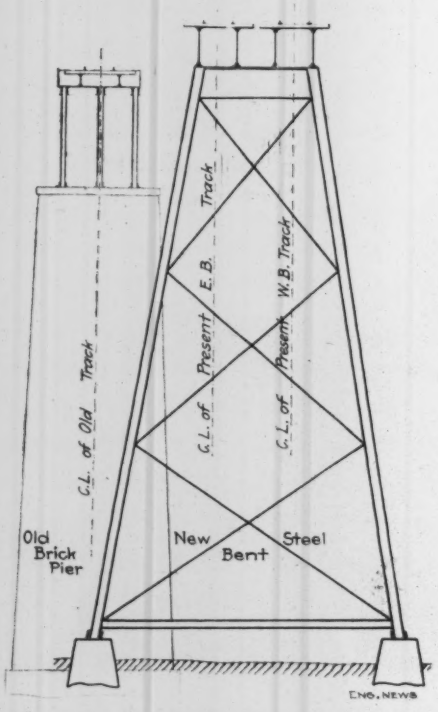


Fig. 3. Comparative Cross-Sections of Old and New Bridges

1870 as a metal structure, the length being reduced to 21 spans, 110 ft. 3/4 in. c. to c. of end bearings. It consisted of deck Fink trusses, built by the firm of Smith, Latrobe & Co., of Baltimore, of Phoenix Iron Co.'s material. The bridge was designed for a live-load of about 1850 lb. per lin.ft. of track; the spans weighed about 70,000 lb. each and were in service until 1886.

Third Bridge—In February, 1886, proposals were invited to strengthen the Fink trusses or construct a new bridge capable of sustaining loads that agree with Cooper's E-24 class, followed by a uniform load of 3000 lb. per ft., the present trusses having only 50% of the strength to safely carry this weight.

Deck Pratt trusses, built of iron, with steel eye-bars, were adopted and were fabricated and erected during the summer of 1886, the contractors being the Edge Moor Iron Co. & Frederick H. Smith. These spans were 12 ft. 2½ in. c. to c. of trusses; 15 ft. 9 in. c. to c. of chords, and consisted of seven panels 15 ft. 9 in. each, making a total length of 110 ft. 3 in. c. to c. of bearings. The ties rested directly on the top chord. The weight of metal per span was about 60,000 lb., and the bridge cost $75,000 erected in place. The record of erecting shows that five of these spans were erected under traffic in 12 days.

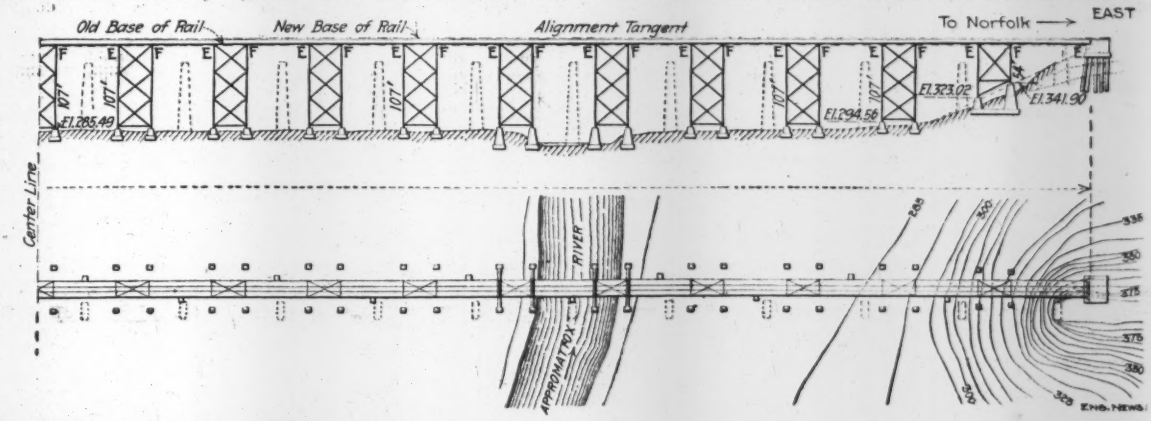
Fourth Bridge—The 1886 spans, resting on the brick piers, carried the increased traffic until 1901, when they were reinforced by the addition of a third truss placed midway between the existing ones and braced to them. A metal floor system was added at the same time, the floor-beams resting on top of the three trusses, the middle truss being designed to carry 48% of the live load from a concentration of four 45,000-lb. axles, 5 ft. c. to c., with a uniform load of 4000 lb. per lin.ft. of track at a 10-ft. interval in either direction. These spans were built by the American Bridge Co.'s Edge Moor plant, the material being soft steel, except the eye-bars, which were of medium steel.

The installation of the third truss and floor system was made under traffic, the grade being raised about 2 ft. to accommodate the new floor system. This latter structure carries safely the present-day traffic, with axle loads as high as 57,000 lb.

Present Bridge

As stated above, the brick piers supporting these spans have shafts in them tapering in size from top to bottom. This condition, together with the vibration from heavy traffic, caused several of the piers to crack near the top. A number of years ago the piers were strengthened by placing bands of steel bars, at about 8-ft. intervals from the top, around the piers, connecting to angles running down the corners. These bands are clearly shown in the views.

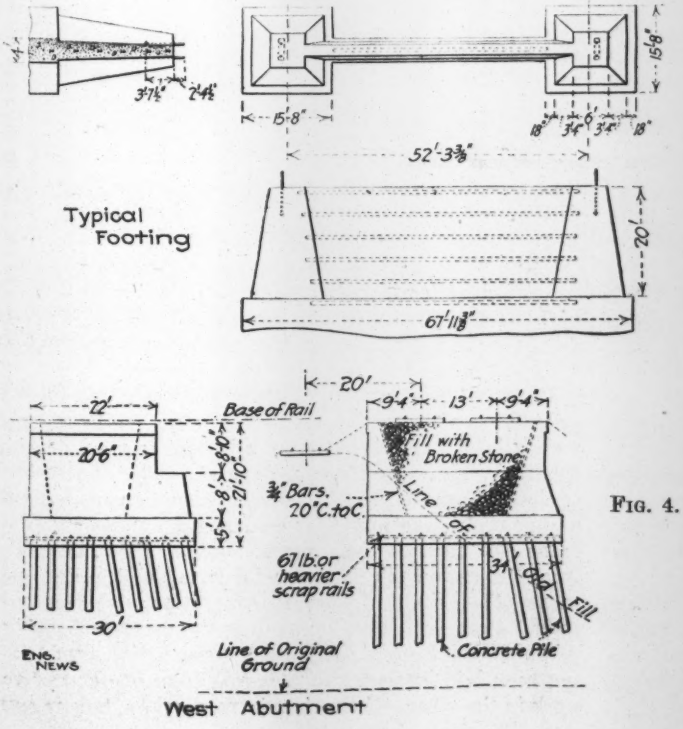
Increased traffic which required the extension of the double-track lines into this territory, together with the appearance of additional cracks in the piers from time to time, caused a study to be made of plans for renewing this bridge for a double-track structure. Surveys were made and plans prepared in the latter part of 1912 for the construction of a double-track steel viaduct, with the center line of the near track parallel to and 20 ft. to the north of the center line of the existing bridge. The cross-section, Fig. 3, shows the relation between the old and the new structure, it being such as to permit the construction of the foundations and the erection of the steel-



High Bridge and (Dotted) the Old Substructure

work without endangering the old substructure, superstructure, or interfering with traffic. Curves, near each end of the bridge, permitted the alignment to be easily adjusted on account of the offset.

Revision in the grades in this territory made it necessary to raise elevation of the base of rail on the new structure about 7 ft. higher than the old, in order to keep within the ruling grade of 0.3 against the eastbound coal traffic.



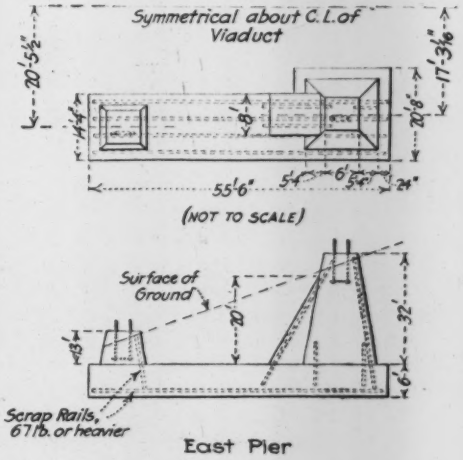


Fig. 4. Details of Substructure of Farmville High Bridge, Norfolk & Western Ry.

The approach embankments at the two ends were made from borrow pits, with short haul, the material being excavated with steam shovel and dumped from light trestles alongside the fill. The materials available for making the fills contained minerals of soapstone and talc formation, which give it a disposition to slip, slides having previously occurred several times in the old embankment from this same cause. This matter was given careful consideration in the preparation of plans for the new structure, with the result that slopes of two horizontal to one vertical were adopted for the fills. As a further precaution the fill was made in three lifts, the material being placed in the outer edges of all slopes with scrapers after dumping from cars and kept in horizontal layers. A ditch was cut about 25 ft. from the foot of the slopes to care for drainage at the base. The approach fill at the west end is about 1800 ft. long; at the east end about 400 ft. long; the two contain 471,000 cu.yd.

The Cummings type reinforced-concrete piles was selected and manufactured at the site and driven by the Cummings Structural Concrete Co., of Pittsburgh, Penn. They are octagonal in section; 17 in. between parallel sides for 20 ft., tapering to 9-in. section, 15 in. from the point; their design is shown in Fig. 5. Heavy spiral reinforcement, ⅜-in. round; ¾- to 1½-in. pitch, is used near the top to resist the blow of the hammer. The piles were molded during the summer and fall of 1913 on platforms underneath the bridge. The reinforcement was placed in the forms and blocked away from the boards so that concrete would properly cover the metal. The top board was left off for pouring the concrete.

The mixture was 1:2:4, with one additional bag of cement per cubic yard of concrete, or about 1.75 bbl. of cement per cubic yard. Broken limestone, size 1 in., was used. Forms were removed after two days and piles were then allowed to cure from 30 to 60 days before driving.

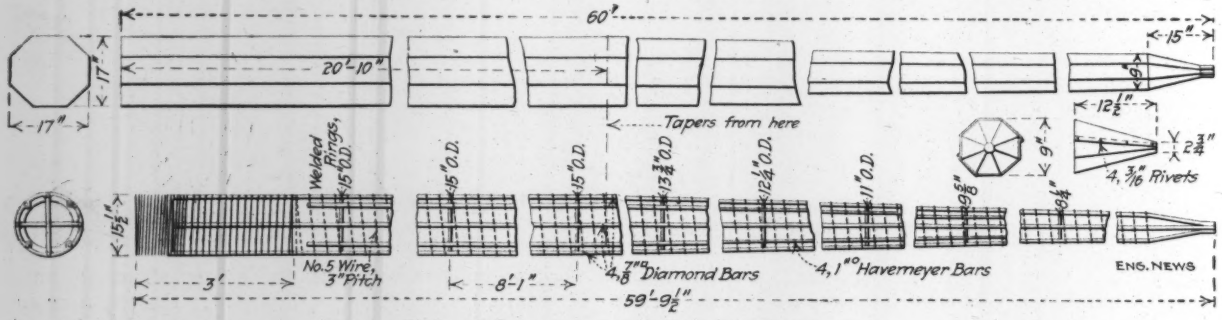


FIG. 5. DETAILS OF CONCRETE PILE

SUBSTRUCTURE—In the new structure there are 43 spans; the end or abutment spans are 90 ft. long; the intermediate spans 72 ft. long, and the tower spans 38 ft. long. The pedestals supporting the towers are 6 ft. square on top, 8 ft. 8 in. square at the base and 8 ft. high, resting on foundation courses 12 ft. 8 in. square. Foundations of this size rest on rock and give maximum load of 7.3 tons per sq.ft. The pedestals adjacent to the river were tied together with concrete struts, reinforced with steel rails, to prevent the accumulation of drift between them.

The pedestals supporting the first tower on either end vary from 20 to 40 ft. high, and the two on either side of the viaduct have a continuous foundation parallel with the track, the upper or higher pedestal is also buttressed with concrete, reinforced with steel rails, to resist the earth pressure from the fill which spills around it. The plan on Fig. 3 shows the arrangement clearly. The foundations adjacent to the river, and practically all others in the low ground, extend to rock, a distance of about 25 ft. below ground.

The abutments, U-shaped, rest on the old fill about 22 ft. below the base of rail. Broken stone fills the space between the wing walls and earth spills around the sides. Steel-rail reinforcement was liberally used in the main and wing walls of the abutments to overcome the tendency to crack from any slight settlement that may occur from shrinkage of the fill.

PILES—Reinforced-concrete piles, cast at the bridge site, were adopted to support the two abutments. Lengths of 40 to 60 ft. were required to make them penetrate to the original ground line. The outside piles were driven on a batter.

A heavy frame piledriver of the ordinary type, having a drop hammer weighing 12,000 lb., was used to drive the piles. A cap, sliding between the leads, covered the top of the pile and held it in line; the blow struck was from 5 to 6 ft. fall. The piles drove hard their entire length, an average of less than two 60-ft. piles being made per day. The penetration was about ¾ in. per blow, the last foot requiring 30 to 35 blows. On this basis the piles have a resistance of 40 tons as calculated by the ENGINEERING NEWS formula, the actual load being 26 tons, including the weight of the pile, which is 6 tons; pressure per square foot on foundation bed, 1.6 tons. Good results were obtained in driving, only a small number being damaged at the top. These were cut off 3 to 4 ft. from the end.

The concrete for the pedestals and abutments was made according to Norfolk & Western standard specifications of 1:2:4 mix, with broken limestone to pass through a 2-in. ring, all dirt and dust being screened out. Good sand was secured by pumping from the river.

PLANT—The contractor's plant was located on the low ground near the west end of the bridge. All material was delivered to this plant by a low-grade track leaving the main line about one mile west. The contractor's equipment consisted of a 10-ton locomotive crane, with orange-peel bucket, used for excavating foundations. This crane also handled the concrete buckets when depositing into forms, the pile hammer to drive sheet piling and many other kinds of work in connection with this contract. Cars were used for hauling the concrete from the mixer to the various pedestals, a construction track having been laid between the pedestals from one end of the work to the other. Concrete mixers, derricks, hoisting en-

gines were supplied to meet the requirements of handling the materials rapidly when needed. Work on the foundations and fills began in May, 1913, and was completed in April, 1914, a total of 10,908 cu.yd. of concrete being placed.

SUPERSTRUCTURE—The viaduct was designed according to the Norfolk & Western specifications of 1911 for a live-load of five 55,000-lb. axles, 5 ft. c. to c., and a uniform load of 5500 lb. per foot on either end at 10-ft. intervals. Double-web cross-girders, 7 ft. deep, were used at the top between the columns to furnish support for the two center girders, the outside girders resting directly on the column cap, diaphragms and stiffeners being used under the bearings of the center girders. The deck girders, spaced 6 ft. 6 in. c. to c., are 7 ft. 6½ in. deep, back to back of flange angles. A standard floor of 8x10-in. bridge ties 12 ft. long, dapped to 9½ in. and spaced 13 in. c. to c., is used, with 6x8-in. guard timbers outside the rails, notched down 1 in. over the ties, and 85-lb. steel guard rails inside.

The columns are made up of one cover-plate $28x\frac{9}{16}$ in.; four angles $6x4x\frac{9}{16}$ in., two webs $24x\frac{9}{16}$ in., double laced on the inside, with 3x⅝-in. lace bars. Columns are built in two sections, with a batter of 2 in. in 12 in.; the base plates are 4 ft. 2 in.x5 ft.x1½ in. thick, and are anchored to the pedestals by two 1¾-in. bolts 6 ft. long, built 3 ft. 6 in. in the concrete with an anchor plate 6 in. wide, placed on the lower ends of the bolts.

The longitudinal and transverse bracing is a double system, capable of taking both tension and compression, and is made up of four angles 5x3½x⅜ in. and 6x4x⅜ in., with single lacing 2½x⅜ in. for transverse bracing, and double lacing the same dimensions for the longitudinal bracing. The bottom struts are made of two 15-in., 33-lb. channels, double laced. Tower bracing in a horizontal plane is made of four angles 5x3½x⅜ in., double laced.

ERECTION—The girders, with the exception of the two 90-ft. spans, were delivered completely riveted in pairs, and unloaded on the east end at grade alongside construction tracks. All the column sections, bracing and cross-girders were delivered and unloaded alongside the con-

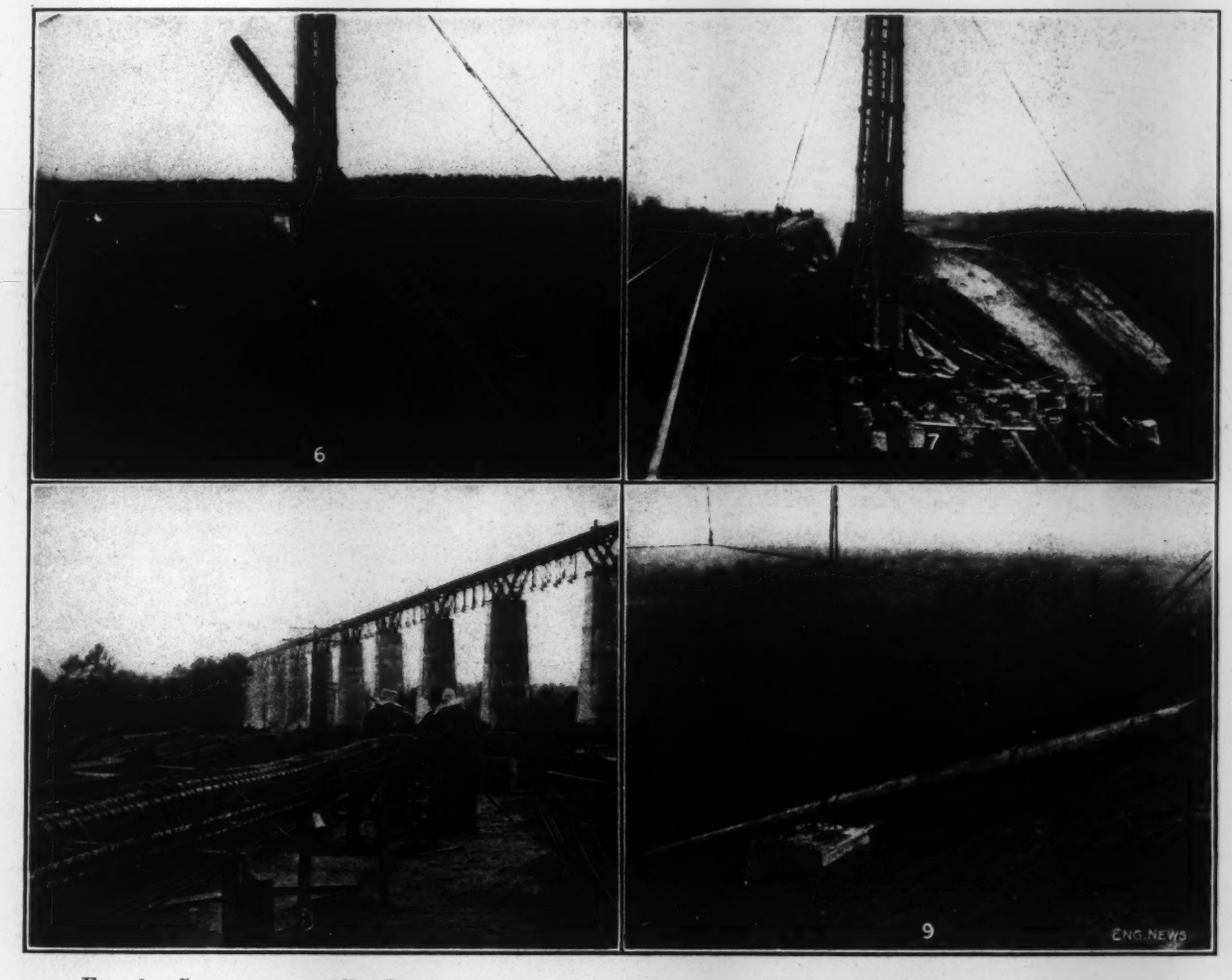


FIG. 6. SLINGING A 60-FT. PILE ONTO THE DRIVER

FIG. 7. THE PILES IN PLACE

FIG. 8. WEAVING THE REINFORCEMENT

FIG. 9. DRAGGING A PILE FROM THE YARD UP TO THE DRIVER

FIGS. 6-9. DRIVING CONCRETE PILES ON FARMVILLE BRIDGE ACROSS APPOMATTOX VALLEY

FIG. 10. ERECTING THE NEW FARMVILLE BRIDGE OF THE NORFOLK & WESTERN RY., ACROSS THE APPOMATTOX VALLEY

struction track running between the pedestals on the low ground.

The bottom section of each bent was erected and all bracing placed, using a 50-ton locomotive crane working on the low track. The top section of each bent was assembled and left standing alongside the bottom section, where it could be lifted to its final position with a 50-ton derrick car, using an 80-ft. boom, and working from the completed viaduct above. The top section of the near bent of each tower was placed, the derrick car standing at the end of the track over the preceding tower. The 72-ft. girder span, weighing 36½ tons, was next run out on a truck to the end of the track, and set off on the ties in a diagonal position; the derrick car next backed off the bridge so the truck could be gotten out of the way, when it returned and placed the span in position. The tower was then completed and the tower span placed.

Erection of the steel began, Apr. 1, 1914; the first span was placed, Apr. 3; last span placed, June 3; riveting completed June 11; floor laid, June 18, and traffic diverted over the new structure, June 22.

As the double track has not reached the immediate vicinity of this bridge, girders for only one track are provided at this time. The total weight of metal is 6,995,628 lb. Metal in the 99-ft. towers, per foot of height, weighed 2325 lb.; per foot of track it weighed 2000 lb. Girders, per foot of track, weighed 900 lb.; total weight of bridge as constructed, 2900 lb. per lin.ft. of track.

ENGINEERS AND CONTRACTORS

The work was started under the direction of Chas. S. Churchill, Chief Engineer; Chas. C. Wentworth, Principal Asst. Engineer, and J. E. Crawford, who, as Bridge Engineer, prepared masonry plans and supervised the design of the steelwork. It was completed under the direction of J. E. Crawford, as Chief Engineer. A. Bruner, Assistant Engineer, was in charge of all double-track construction and W. W. Vail was Resident Engineer on the work.

The Virginia Bridge & Iron Co., of Roanoke, Va., designed the steelwork, furnished and erected the structure.

Messrs. W. W. Boxley & Co., contractors, of Roanoke, Va., built the masonry and approaches.

A Fireless Locomotive for use at powder magazines and explosives storehouses has been supplied to the British navy department. It resembles a small four-wheel locomotive without a smokestack, the cylindrical tank containing some water and being charged with steam at high pressure. The steam passes through a reducing valve before reaching the valves and cylinders which are placed at the rear end of the engine. One charge is sufficient for several hours' work at steady hauling and for a still longer time in switching service, while with a pressure as low as 15 lb. the engine can run to the charging station, which is outside the danger zone. To further reduce the chance of danger from sparks the brakeshoes, bumpers, etc., are fitted with special facings. The engine was built by Andrew Barclay, Sons & Co., of Kilmarnock, Scotland.

Asphaltic Concrete Surfacing for Suburban Roads, Chicago, Ill.*

Suburban roads about Chicago have cost approximately 20c. per sq.yd. per year to maintain, and at no time has there been a road with a smooth wearing surface. In three years, the cost of upkeep on each macadam road, it is estimated, will approximately equal the first cost of an asphaltic concrete surface, 2 in. thick, which can be placed by municipal forces for between 50 and 60c. per sq.yd.

The low cost of this work is largely attributed to the planning of the work, provision of modern municipally owned equipment and to the efficiency of the organization. It is estimated that the macadam road covered by 2 in. of asphaltic concrete will not require any repairs for five years and that with reasonable repairs, this pavement will last 20 years.

The repair and maintenance of roads and pavements in Chicago is under the general direction of the Superintendent of Streets and under the immediate supervision of the general foreman in charge of the gangs, which are distributed in different sections of the city. The planning of the work is done by a number of civil engineers who report directly to the superintendent in charge of street repairs. In all cases, the street-repair foreman in charge of gangs works in conjunction with the superintendent of the ward in which the work is being done.

All asphaltic material required for the repair of pavements and roads is manufactured by the Bureau of Streets at the municipal asphalt plant and by a portable asphalt plant. These materials are transported by municipally owned vehicles, and the work is done by the organization which is recruited from civil-service lists.

*Mainly from an article prepared for the Journal of the Chicago Automobile Club, by J. L. Jacobs, Efficiency Engineer In Charge, City Service Commission, Chicago, Ill.

The portable asphalt plant can be moved about on the outskirts of the city where it is proposed to resurface macadam streets with bituminous top. This reduces the cost of long team haul, which is usually a large factor in the cost of repairing pavements. Under this arrangement material is received on the track beside the portable plant where it is manufactured and transported to the job by 5-ton motor trucks, recently purchased by the Bureau for this work.

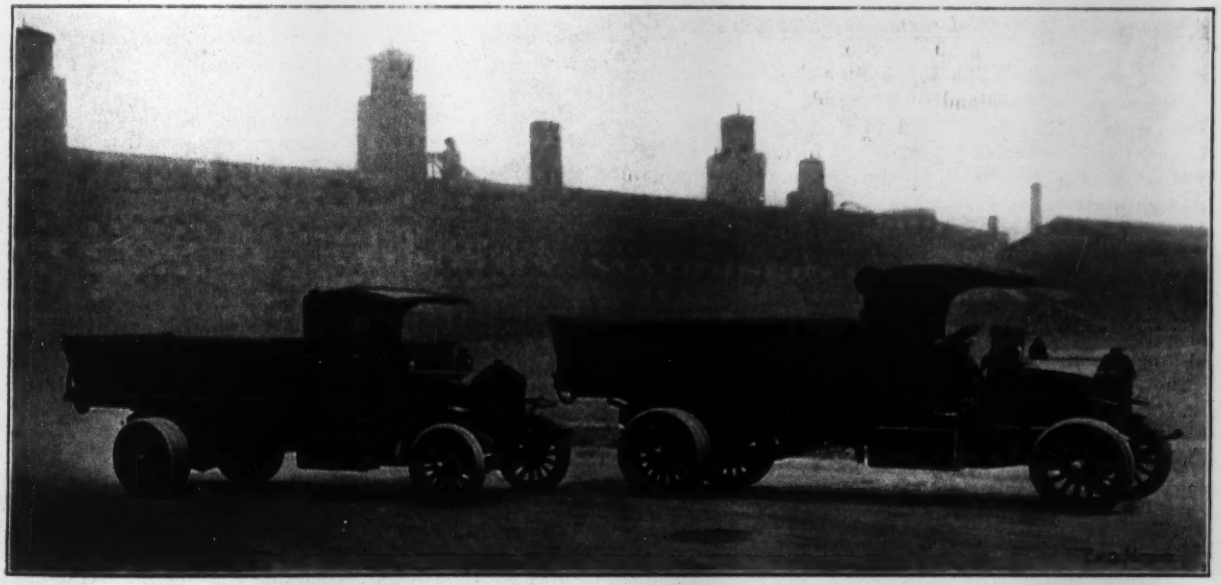
Appropriations have been provided and specifications prepared for another portable plant which is to be used to utilize old asphalt top, which heretofore had to be transported and discarded at a large cost. The old asphalt will be brought to this portable plant, crushed and reheated together with new material, and taken back to the job to be used as binder. In this way the expense of long haul and waste of old material will be materially reduced and the capacity for new work will be proportionately increased.

In order to keep all the equipment in good condition, shops and storage sheds have been provided, and all the equipment is overhauled during the winter months, so there is little or no breakage or delay on account of run-down equipment during the working season.

Careful cost records are kept of the repairs to various types of pavement, and yearly averages are published in the report of the Public Works Department.

Engineering Vernacular as Interpreted by a Lawyer—An erecting contractor, who was building part of one of the great suspension bridges across the East River in New York, visited a law office and undertook to explain his troubles to a law student in the office. The would-be lawyer made notes and, confessing that he did not thoroughly understand the situation, made a report in substance as follows:

A Traveler was on the lower cord of the approach span of the Williamsburgh Bridge and was sustained by four Guys. Each Guy was held fast by two dogs, after passing through an opening in the tail piece. Tony, an Italian, had just been sent out and had given two of the dogs a fresh bite on the longest Guy, when they let loose, one of the Guys slipped, and the Traveler tipped over and threw the counter-weights into the air, and they went down on the street below and killed a man.—From an address by John C. Wait before the Civil Engineers' Society of Cornell University.



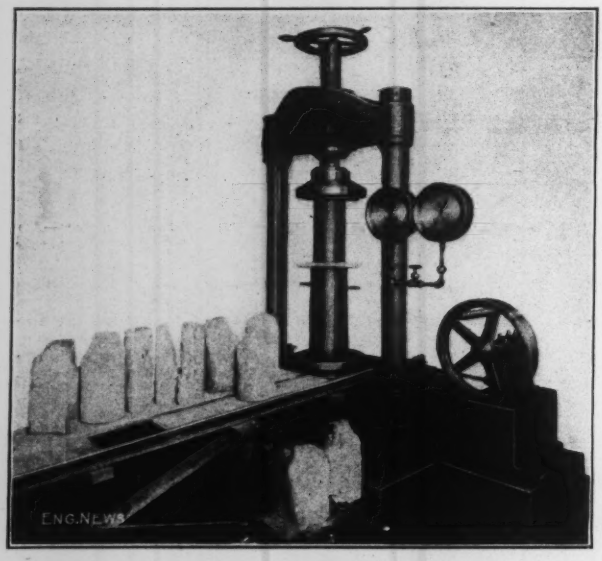
TYPE OF "PIERCE-ARROW" MOTOR TRUCK USED FOR HANDLING HOT ASPHALT PAVING MIXTURES, CHICAGO, ILL.

Field Tests of Concrete for Municipal Work at Kansas City, Mo.

By E. S. Wallace*

The early tests on concrete used in municipal work at Kansas City, included the usual tests on cement and sand but the tests on the large aggregate were confined to some very primitive investigations which consisted in making comparative mixes with different aggregates, placing them in similar molds, and at the end of certain periods, breaking them by striking repeated blows with a hammer. The quality was determined by the manner in which the concrete resisted the impact of the blows.

At best, this was a most unsatisfactory test and about a year ago the city started a systematic method of testing aggregate, and in fact, concrete, by taking samples of all concrete on the job and subjecting the samples to



200,000-Lb. Compression Machine for Testing Concrete Cylinders Used by Kansas City, Mo.

compressive tests through periods up to one year. The city now has a 200,000-lb. hydraulic compression machine and one dozen 8x16-in. cast-iron cylindrical molds turned to a true surface. These molds are taken to the place where concreting is in progress and without the previous knowledge of the contractor, the concrete is caught in a bucket as it is discharged from the mixer, and filled into the molds. No two molds are filled from the same batch, which possibly accounts for the wide variation in compressive strength of sets of cylinders of the same age. A ⅜-in. gas-pipe flattened at one end is used to spade the contents in the molds, and no tamping is done further than to make the cylinders as near like the concrete as it is deposited in the work as possible. At the time, the style of mixer, weather conditions, kind and class of materials, etc., are noted.

The molds are allowed to stand until the concrete has set; usually from 12 to 24 hr. before removing to the laboratory, where the forms are taken off and the concrete cylinders are placed in a storage tank, which is kept filled with water at a temperature of about 70° F. The storage tank is a rectangular-shaped concrete box, 3x22 ft. in plan, 2 ft. 4 in. deep; it has a floor drain at lowest point for drawing off the water and a ¾-in. perforated steam pipe runs the entire length of the bottom for heating the water in cold weather, as it is outdoors in an areaway.

The accompanying view shows the machine with a cylinder in position for crushing, and several tested cylinders in the foreground showing manner of fractures. It will be noticed that the openings between the uprights and the distance between platens of the machine is much greater than necessary for testing concrete cylinders, but it was so designed in order to make crushing tests (known as the knife-edge test) on sewer pipe up to and including 27 in. in size. The cast-iron stool saves using a long and cumbersome adjusting screw.

When making tests, a rubber plate about ¼ in. thick is placed on the stool, the cylinder is set upon this, and another rubber plate placed on top. These plates are used to even up any irregularities there may be in the end surfaces of the cylinders and are believed to be superior to plaster of paris. The top platen, which is suspended from the adjusting screw on a ball-and-socket joint, is now screwed down firmly, and the load applied. As will be noticed, the machine is in reality a hydraulic jack, operated with a three-plunger hand-power pump, and the load is applied from underneath on the bottom platen.

Two gages are used; one is graduated in 50-lb. readings up to 10,000 lb., and the other is graduated in 500-lb. readings up to 200,000 lb. Each gage is equipped with a marker hand which registers the highest ultimate load attained. The load is applied at the rate of about 35,000 lb. per min.

As a rule the cylinders crush off at angles of about 45° on either top or bottom end, though some of them split from end to end into two or more prismoids. Observation has shown that concrete is quite green at 7 days, and is still improperly cured at 28 days. It would seem that much better results could be obtained from concrete if it were not subjected to live loads until after the 28-day period.

There is given also herewith a table of compression tests. As before noted there is a wide variation between tests of the same age, and complaint might be made against these discrepancies, but the writer has only attempted to make tests as near like the material itself under working conditions as is possible, and no effort was made to obtain a concrete that would show uniformity in testing.

Contractors on city improvements are permitted to use any brand of cement they choose so long as it complies with the city's specifications, which are those of the American Society for Testing Materials, but it so happened that only three brands are shown in the table; they are standard brands of this district. A careful study of the chart will convince most readers that conditions and workmanship play by far the most important part in the manufacture of good concrete, and irregularities shown in these tests cannot be attributed to the cement.

In Test 12, cement "A" shows the highest results obtained, while Test 14 with the same brand of cement, same mixture, and same style of mixer, but by different contractors, shows the lowest. Test 5, cement "B," with

*Inspector, City Engineering Department, Kansas City, Mo.

TABLE SHOWING COMPRESSIVE STRENGTHS OF FIELD-MADE CONCRETE, KANSAS CITY, MO.

No.	Date	Contractor	Class of work	Style of mixer	Brand of cement	Mixture	Consistency	Aggregates	Pounds per square inch 7 days	28 days	90 days	180 days	Weather	Temp. °F.	
1	5-28-13 p.m.	A	Concrete pav. alley.	Continuous	A	1 : 3 : 5	Quaking	2-in. native limestone—clean, hard and uniform in size.	815	934	3400	1770			
									715	964	1600	2020	Fair	90°	No aggregates failed in 7- or 28-day tests,—fractured freely in 90-, 180- and 360-day tests.
									994	1385					
									Avge. 841	1094	2500	1895			
2	6-18-13 a.m.	B	Concrete base for brick paving.	Batch	C	1 : 3 : 6	Wet	Old concrete base crushed and screened about 2-in. size—clean.	520	695	1050	950			
									460	805	1070	1180	Fair	80°	Old concrete mortar failed in each test—limestone aggregates, contained in old concrete, did not fracture in any test.
									520		950				
									Avge. 500	750	1023	1065			
3	8-28-13 p.m.	E	"	Hand mixed	B	"	Quaking	Old concrete base. Hand crushed, not screened.	300	560	960	1050			
									480	720	1150	1250	Fair	85°	Old mortar failed in each test. Limestone aggregates did not fracture in any test.
									280	740	900	1100			
									Avge. 353	673	1003	1133			
4	9-24-13 p.m.	G	"	Batch	B	"	Wet	Old concrete base. Hand crushed and forked—not very clean.	340	360	550	900			
									200	250	540	750	Cloudy	80°	On account of damp weather but little sand or dirt was separated by forking. No aggregates failed.
									230	230	420	710			
									Avge. 257	280	503	787			
5	10-6-13 a.m.	K	"	"	B	"	Very wet Sloppy	"	780	460	700	1780			
									600	1050	802	1780	Cloudy	70°	Aggregates were fairly clean for material of this class—mix. A few of the rock in old concrete aggregates failed in 180-day test.
										410	1160	1500			
									Avge. 690	640	887	1687			
6	10-8-13 p.m.	I	"	"	A	"	Quaking	2½-in. native limestone, about ⅓ of which was old curbstones. Hand crushed, clean.	240	590	950	1850			
									320	590	1400	1040	Fair	75°	Aggregates fractured in 180-day test.
									310	450	810	1900			
									Avge. 290	543	1053	1597			
7	10-30-13 p.m.	N	Base for asphalt	"	A	"	Wet	Old concrete base. Hand crushed. Extra clean.	260	860	1210	1220			
									250	760	1290	1200	Clear	45°	Temperature dropped to 30° during the night and concrete was quite soft when brought to laboratory.
									240	1150	1280				
									Avge. 250	907	1260	1210			
8	10-13-13 a.m.	B	Base for brick paving blocks.	"	C	"	Wet, sloppy	Old concrete base. Machine crushed and screened clean.	670	900	1490	1660			
									860	1050	1480	1900	Fair	65°	Aggregates were quite uniform in size and clean.
									660	1040	1240	1730			
									Avge. 730	997	1403	1763			
9	8-25-13 p.m.	C	Monolithic sewer	"	B	1 : 2 : 4	Wet.	1-in. native limestone. Crusher run. Clean.	260	400	1030	1200			
									200	1280	650	1300	Clear	100°	Aggregates fractured in 180-day test.
									380	330	910	1450			
									Avge. 280	670	863	1317			
10	8-28-13 a.m.	D	Sidewalk	"	A	"	Quaking.	"	300	850	1700	1650			
									480	660	980	2000	Clear	90°	A few aggregates failed in 180-day test.
									280	600	930	1450			
									Avge. 353	737	1203	1700			
11	8-29-13 p.m.	F	Reinforced concrete sewer pipe.	"	A	"	"	¼-in. native binder. Crusher run.	740	1300	2500	1960			
									700	1220	1850	2750	Clear	100°	This cement was more thoroughly mixed than is commonly practiced. Aggregates fractured freely in 90- and 180-day tests.
									600	1280	2140	1900			
									Avge. 680	1267	2163	2203			
12	9-30-13 a.m.	I	Curb.	Continuous	A	"	Dry.	¼-in. native limestone. Crusher run, not very clean.	950	1840	3020	2870			
									1030	1440	2050	2730	Clear	80°	Aggregates fractured in 28-, 90- and 180-day tests.
									1010	1680	2300	2640			
									Avge. 997	1657	2685	2747			
13	10-2-13 a.m.	J	Sidewalk.	"	C	"	Not well mixed. Very dry.	¼-in. native binder. Screened on No. 10 screen.	520	1230	700	1240			
									740	773	802	750	Clear	80°	Mixture was so dry as to be impossible to make a dense concrete. Water oozed from cylinders when load was applied. No aggregates fractured in any of these tests.
									600	973	1160	960			
									Avge. 640	991	887	983			
14	11-20-13 p.m.	O	Curb.	"	A	"	Very dry.	¼-in. native grit, coated with yellow clay.	280	470	580	740			
									340	360	530	720	Clear	70°	The cylinders were porous and crumbly. No aggregates fractured in any of these tests.
									300	380	480	780			
									Avge. 307	403	530	747			
15	10-24-13	L	Concrete paving.	"	B	1 : 2½ : 4½	Wet.	2-in. native limestone—clean and uniform in size.	400	660	1040	1250			
									280	680	1040	1600	Clear	65°	Temperature dropped to about 35° and it was about 36 hr. before concrete was hard enough to remove from the moulds.
									340	630	1040	1080			
									Avge. 353	657	1040	1163			
16	10-24-13 a.m.	M	"	Batch	A	"	"	2½-in. native, clean and uniform in size.	310	500	1000	500			
									170	620	880	1340	Cloudy	40°	Temp. dropped to 24° during the night and the cylinders froze quite hard—were allowed to remain in molds 24 hr. in laboratory till concrete thawed out and took initial set.
									250	530	960	1180			
									Avge. 280	550	947	1007			
17	11-12-13 p.m.	H	"	"	A	"	"	2-in. native, clean and uniform in size.	250	1480	2150	2720			Sand was black and dirty looking. Aggregates fractured in 28-, 90- and 180-day tests.
									500	1620	1440	1530	Cloudy	65°	
										950	2500				
									Avge. 375	1350	2197	2125			
18	11-20-13 p.m.	H	"	"	A	"	Quaking.	"	340	800	1550	1440			
									460	550	1400	1320	Clear	75°	A few aggregates fractured in 90-, 180- and 360-day tests.
									350	650	930	1400			
									Avge. 383	667	1293	1387			

Missouri River sand used in samples 6, 7, 8 and 10. Kaw River sand used in all others. Kaw River sand is coarse and sharp. Voids in loose sand = 32.2%. Compacted 29.9%. Missouri River sand is quite fine. Voids in loose sand = 31.6%. Compacted 27.0%. Quality of limestone aggregates in these tests was quite uniform. Aggregates marked "Crusher Run" include, from size indicated, to and above a No. 10 screen. Old concrete base aggregates referred to in tests Nos. 2, 3, 4, 5, 7 and 8 were made from Natural cement, the old mortar failed in each test, including the 7-day. Tests indicated that limestone aggregates seldom fail under 1400 lb. pressure per square inch. 360-day test on No. 1 (2070 and 2940 lb.)—avge. 2555 lb. 360-day test on No. 2 (1530 and 1410 lb.)average 1470 lb.

a 1:3:6 mix and an aggregate of broken up old concrete made from natural cement shows an ultimate load of 1687 lb. per sq.in. in 180 days while Test 9, with the same brand of cement, a 1:2:4 mix and a ¾-in. native stone aggregate shows an ultimate load of only 1317 lb. per sq.in. at the same age.

A person may easily determine the thoroughness of which a batch of concrete is mixed by its looks, but our tests have demonstrated the foolishness of anyone attempting to judge its qualities by observation. A well graded sand and aggregate with hardly enough cement to hold them together, may be mixed so thoroughly as to look quite rich, while a batch containing plenty of cement, but with a nongraded aggregate, and not so thoroughly mixed may appear quite "poor and lean." When the writer was taking sample No. 12, by observing the mixture, he considered it very poor and noted it accordingly, and No. 18 was noted as being very good, but actual tests have given results which were exactly the opposite, showing the uncertainty of trusting to observation alone.

It is believed that the 8x16-in. cylindrical molds are the best suited where aggregates of 1 in. in size or over are used, as this size permits the large pieces of aggregates to chamber well, and eliminates the arching effect which would leave voids.

There are no specifications requiring tests of the above named character, for the work, so far as has been experimental, and for the purpose of collecting data. The results obtained in most cases have been different from what were expected, and in some instances have been a surprise, yet it is felt that they have been more than paid for the trouble and expense.

The Sale of Bottled Mineral Water in the United States in 1913 amounted to 58,000 000 gal., valued at over $5,600,000. This was a decrease of 7% in quantity and 15% in value compared with 1912, and the 1912 production was less than that of 1911. According to a press bulletin issued by the U. S. Geological Survey, R. B. Dole, of the Survey, believes that the decrease is "traceable to the decrease in demand for bottled spring waters consequent on improvements in municipal water-works."

The Present Status of Small Cube Granite Block Pavements

As noted in the article on "Recent Developments in Granite Block Paving" in ENGINEERING NEWS, of Aug. 13, 1914, there is a growing tendency to reduce the size of granite paving blocks. Where specifications formerly called for blocks 7 and 8 in. deep, the present practice is to specify 4½ and 5 in. What is the minimum depth and size of blocks?

For durability 3½ in. of granite answers every purpose of 5 in., since wear is seldom appreciable on the surface of granite blocks in any pavement. For rigidity, the present practice is to depend upon a good portland cement concrete foundation and not upon the wearing coat, whatever its type. For a foothold for horses' feet and non-slipperiness the smaller the blocks and their arrangement in non-regular courses, the better the practical results. For cheapness, if the small blocks can be produced by machine at less cost than the larger hand-shaped blocks, the less stone will be required, and the item of freight and haulage, which is a very large item, will be much reduced. As to noisiness the smaller stone surfaces with asphaltic filler between appear to be less noisy than the present standard-size block pavement.

FIGS. 1 AND 2. VIEWS OF "DURAX" GRANITE BLOCK PAVEMENT, BROOKLYN NAVY YARD, NEW YORK CITY

In the matter of adopting small granite cube pavements foreign practice has been much in advance of American practice. Small granite cube pavements have been used on German streets and roads for more than 30 years. They have been extensively used in many English and European cities. About 1,500,000 sq.yd. have been laid in Chile and about 1,000,000 sq.yd. in Argentina, South America, where a large percentage of the blocks are imported from Sweden and Norway as ballast for westbound ships.

In the United States, the first small cube or "Durax" pavement was laid in the Brooklyn, N. Y., Navy Yard in 1913 (Figs. 1 and 2). Since then pavements of this

type have been laid in Louisville, Ky., Cincinnati, Ohio, Columbus, Ohio, Troy, N. Y., Danville, Va., Salisbury, N. C., and contracts are pending in several of our large Eastern cities. During the year 1912 about 2,300,000 sq.yd. of "Durax" or, as they are known in Europe, "Kleinpflaster" pavements were laid.

"Durax" Pavement Construction—The common European practice for city streets is to bed the blocks evenly on ¾-in. of stone chips, or sand, on a concrete foundation, the thickness of which is made to vary with the kind and density of traffic to be carried. After the blocks are placed, the joints are filled with clean stone chips and rammed to a solid bearing. After ramming, the joints are flushed with stone chips and poured full of a bituminous mixture heated to about 300° F. Enough mixture is used to fill the joints completely, making the pavement impervious. Cement grout filler may be used, but in this case the pavement is less elastic and more noisy. Finally, about ½-in. of sand is spread on the surface of the finished pavement and left to work into the joints.

On roads in suburban districts an old macadam base has been used successfully for a foundation. On these roads the blocks are laid on a 1-in. layer of sand and the joints are filled with sand only, after which the surface is wetted, rammed and rolled with a 5-ton roller. This gives a surface of the nature of a macadam road, but owing to the larger and hand-placed stone wearing coat it is, of course, more durable under dense or heavy traffic.

Fig. 3. Oyster-Shell Pattern for Laying "Durax" Blocks

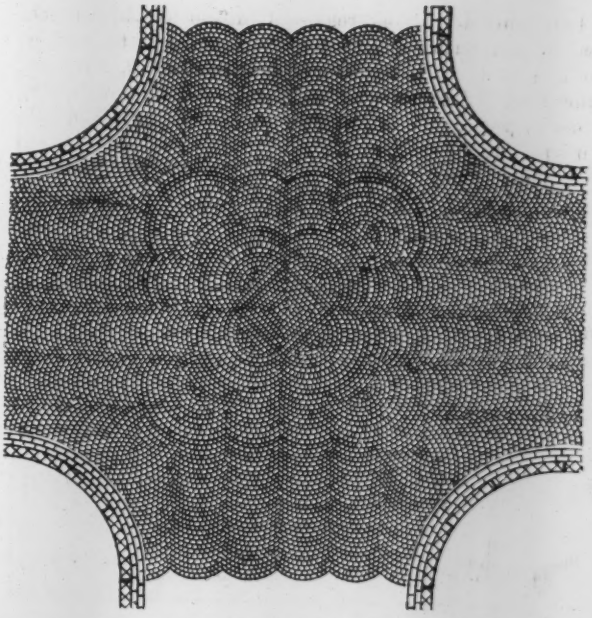


Fig. 4. Street Intersection Pattern, German Design

Ordinarily, the blocks are not laid in straight courses, but in an oyster-shell pattern (Fig. 3), that is in concentric interlocking segments of circles or diagonal courses. The advantage claimed for this method is, that the opposite wheels of a vehicle are not likely to be on the joints between stones at the same time, which reduces the jar and the subsequent wear and tear on the vehicle. A further advantage claimed is that this arrangement better distributes the wear on the pavement.

American Experience

The specifications for the Brooklyn Navy Yard pavement provided that the "Durax" blocks should be irregular cubes, the edges to vary from 3½ to 2¾ in.; that the blocks should be laid in close contact with no joint exceeding ½-in. and in concentric interlocking segments.* The blocks under these specifications, were to show a compressive strength of not less than 30,000 lb. per sq.in., and a coefficient of wear of not less than 15 (French standard).

In Danville, Va., the cubes were of medium-grained granite 3½ in. to 2¾ in. in size. P. F. Brown, Assistant City Engineer, furnishes the following statement:

We used machine-made "Durax" cubes (furnished by the Harris Granite Quarries Co., of Salisbury, N. C.). The pavement is bedded on a ¾-in. sand cushion upon a 5-in. concrete base, and is bonded with an asphalt paving filler (Texaco No. 39) using 2 gal. of filler per sq.yd.

This pavement was laid by the city forces, using our regular block pavers. The cubes are laid in straight lines across

*See article in "Engineering News," Mar. 27, 1913, p. 629, "A Comparison of Recent Bids for Various Types of Paving in the New York Navy Yard," by Walter H. Allen.

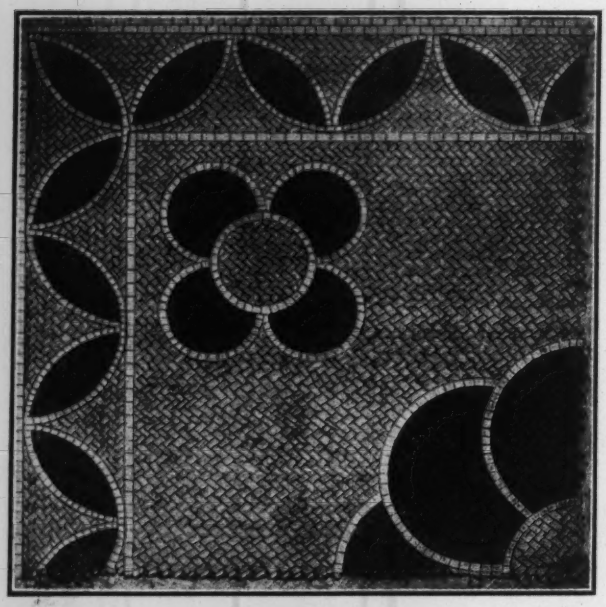


FIG. 5. STREET INTERSECTION OR COURT-YARD DESIGN; SHOWING POSSIBILITY OF MOSSAIC WORK

the street, at right angles to the curb. The pavement cost $2.60 per sq.yd., of which the concrete base (1:3:6 mixture) cost $0.74 per sq.yd. It might be of interest to note that the labor account for laying "Durax," including preparing sand cushion and applying asphalt filler was $0.24 per sq.yd.

This is an excellent pavement, giving a smooth waterproof surface, comparatively noiseless and free from dust; and on account of the smallness of the blocks furnishes an excellent foothold. We are well pleased and feel confident that it will prove most satisfactory.

The "Durax" blocks are generally more irregular in shape than the standard granite blocks now required for first-class city pavements, but being machine made can be turned out cheaper than the more perfectly shaped hand-cut blocks. Their cubical shape also gives the advantage that the most suitable one of the three depths may be used, which makes possible a pavement of fairly uniform thickness in spite of the irregular faces of the block.

REPAVING WORK—The 3½-in. granite cubes are particularly adaptable for repaving city streets which have a good foundation left over from the former surfacing. The depth of the cubes being practically the same as wood blocks, brick and asphalt surfacing, the new surface may be placed without removing the old foundation or changing the grade.

POSSIBILITIES OF THE SMALL CUBE PAVEMENT—There are two points in pavements laid with small cubes which stand out pre-eminently. One the ease with which warped and irregular surfaces may be paved in well rounded and pleasing curves, as is illustrated in Figs. 1 and 2 of the Brooklyn Navy Yard work. The other point is the possibility of artistic mossaic work in courts, public squares, etc., using different colored stones, as illustrated in the diagrams, Figs. 4 and 5, from the German journal "Die Strasse." While engineers, and perhaps especially highway engineers, are seldom interested in the esthetic points of construction, this is a point which will appeal to architects and city planning experts.

The small granite cube type of pavement is not patented. As noted before, it has been used in Europe for many years; but the extensive increase in its use in recent years is due largely to the method of manufacture by the Wern stone-cutting machine, which was fully described in ENGINEERING NEWS, Mar. 28, 1912, p. 614.

The First Water-Works in America are said to have been constructed in 1754 by Hans Christopher Christiansen to supply the Moravian settlement at Bethlehem, Penn. The water-works included a pump of lignum-vitæ, of 5-in. bore.

FIG. 6. "DURAX" GRANITE BLOCK PAVEMENT CONSTRUCTION IN BUENOS AIRES, ARGENTINA; IMPORTED SWEDISH BLOCKS

Constructing a High-Pressure Gas Line to Louisville, Ky.*

By Lewis S. Streng†

The construction of a 12-in. steel pipeline for carrying natural gas from Inez, Ky., to Louisville—a distance of 180 mi.—presented many difficulties. The work was commenced Aug. 8, 1913, and completed and tested on Mar. 12, 1914.

The unusual difficulties of the undertaking were due largely to the nature of the country. The work on the Louisville end was simple compared to that in the mountains. Delivery of material was a problem. In some places, the nearest railway was 40 mi. distant, and the wagon roads hardly deserved the name. A team would start from Paintsville with two lengths of pipe, the load amounting to less than a ton, and return light, taking three days for the round trip. After 150 teams had made several trips, the roads were in miserable condition, requiring a repair gang to keep them passable.

On the Louisville section, much foreign labor was employed, but not back in the hills. The mountaineers drove the foreigners out of the country. The mountaineers were splendid workmen. They would walk four or five miles before daybreak, do a day's work and go back home with a lantern.

The steel pipe used was 12 in. inside diameter, 3/8 in. thick, weighed 45 lb. per ft., and averaged 20 ft. to the length or "joint" as it is called. Each joint was tested at the mill at 1000 lb. per sq.in. pressure and carefully inspected for defects. The couplings used were steel forgings of the familiar Dresser type, consisting of a central ring, two rubber-compound gaskets and two followers drawn up by steel bolts. Screw pipes were used in all river and creek crossings, with leak collars with rubber gaskets placed over each of the screw couplings, and over all this was bolted a heavy split-iron casting, or river clamp. Under all rivers, two lines were laid with valves and Y's at either end.

At railroad crossings, the pipe was placed in a 20-in. cast-iron casing, fitted with valves, on either side of the right-of-way. Eventually, a second line will be laid, with provisions for cutting out any section desired. In case of trouble, the check valve will prevent the escape of gas from the Louisville end.

Testing—The line was tested as laid before any of the couplings were backfilled. For testing purposes, two temporary compressor stations were erected, one on the Kentucky River, about 7 mi. above Frankfort, and the other on Mud Lick in Johnson County. The line was pumped up to 350 lb. per sq.in. pressure, and then carefully inspected for leaks. The excellent results obtained by this method were shown by a 48-hr. acceptance test of the whole line last February. At 350 lb. per sq.in. pressure, the leakage in 24 hr. was less than 1/4 per cent.

Construction Procedure—The construction force was organized in two divisions, one for the mountains and one for the Blue Grass, each in charge of a general superintendent. The headquarters of the mountain division was at Paintsville, with a territory to be covered of about 65 mi. The other division was about 115 mi. long, with headquarters at Lexington. Each division had two large gangs, with full camp equipment. These gangs started at the compressor stations and worked toward the ends of the divisions. In addition, each division had a special gang which installed the river, creek and railroad crossings. After more than half the work was finished, another gang was organized to hasten the completion of the work. The number of men in each gang varied from 200 to 500.

During the preceding summer, a survey had been made, and most of the right-of-way obtained. Stakes were set every 200 ft. and the first job of the construction gangs was to clear the right-of-way from 16 to 20 ft. wide. The digging crew then roughed out the trench, throwing the earth to one side; the other side being left clear for pipes. Where rock was encountered, a drill gang came next and the way blasted. After them came the graders, who trimmed the trench to proper dimensions, normally 18 in. wide by 36 in. deep.

Changes of grade up to perhaps 5° could be made in the couplings; but in general the straight pipe was laid at an even grade, and fire bends made where necessary. A small gang went ahead inspecting the pipes, dressing off any roughness at the ends, and making the necessary bends.

The pipe was then laid on skids over the trench and the couplings fitted. After a section had been finished, the lengths varying from 100 ft. to a half-mile, according to topography, the gang came back and lowered the pipe into the ditch.

Lastly, followed the backfilling gang. On level stretches, teams with drag-scrapers were used. On the slopes, breakers were built to hold the fill and leave the couplings exposed. Following the laying of the sections in the trench and their testing out, a small gang covered the couplings and trimmed up the fill.

The task of hauling material was a serious problem, as has been described, but that of stringing the pipes along the right-of-way was even more difficult. At convenient locations, the pipe could be unloaded where desired, snatch teams being added as required. Frequently, however, block and tackle were required. Five or six teams were sometimes needed to move a single joint.

On some of the trenching, gasoline-driven drag-line ditching machines were used. In good going, they could dig 2000 ft. of 18x36-in. trench in a day; but they were useless on grades, or where rock was encountered. They traveled very slowly and lost much time by necessary detours.

The best day's work reported by any of the gangs was about one mile of ditch. Occasionally, a pipe gang would report as high as 6000 ft. of pipe laid in a single day; in the rough country, 750 ft. was an average day's work. At times, when practically the whole trench was through rock, a gang of 200 men could not make more than 200 ft. per day.

*Extract from a paper presented before the Engineers' and Architects' Club of Louisville, Ky.

†Louisville Gas & Electric Co.

An Argument for Municipal Engineers—As street paving is without doubt one of the most important municipal functions, it is the plain duty of city officials to keep the pavements in good condition. We believe that if funds are not available for all of the city activities which are being assumed, that it becomes necessary for the city, as it would for a business firm or individual, to retrench; retrenchment should curtail those activities which are not primarily functions of municipal government, in order that those functions which are clearly the duty of the municipality may not be impaired. —Report of the Municipal Committee of the Cleveland Chamber of Commerce on "Cleveland Pavements," April, 1914.

Diaphragm Measurements of Water in Open Channels

By C. R. Weidner*

The measurement of large quantities of water, with a reasonable degree of accuracy, is a problem whose solution is often a laborious, expensive and time-consuming procedure. A device which will simplify the operation without decreasing the accuracy of the measurement was invented about nine years ago by Prof. Erik Andersson, of the University of Stockholm. It is termed the diaphragm or moving-screen method of gaging water. The method has been used in Europe with considerable success for the testing of turbines. Its accuracy, however, was not determined until recently, when the Swiss Bureau of Hydrography conducted a series of experiments for that purpose. The results of these tests showed a high degree of precision, and this article has been prepared with the object of bringing the device to the attention of American engineers.†

The diaphragm method of gaging is essentially a modification of the float method, in which the velocity is integrated over the entire area; its use is, however, restricted to the measurement of water flowing in a channel of uniform cross-section. The apparatus consists of a diaphragm suspended from a car, which runs on a carefully lined and leveled track laid on the canal walls. The frame of the car and diaphragm is built of steel tubing or light angle iron; the wheels of the car are usually made of aluminum and run on ball bearings. The diaphragm frame is covered with oiled canvas or some light metal and swings about a horizontal axis in the direction of the current, so that it can readily be immersed or withdrawn without causing an appreciable rise in the elevation of the water surface. When a measurement of the velocity is being made a clutch holds the diaphragm in a vertical position. Figs. 1 and 2 show the rating canal and diaphragm in use at the J. M. Voith turbine-testing station, at Heidenheim, Germany. The cross-section of an installation at Notodden, Norway, is given in Fig. 4.

The method of procedure in obtaining a gaging is quite simple. A measured distance is first laid off along the canal walls. The diaphragm is then dropped into the stream at a point far enough upstream, so that it will have obtained uniform motion by the time it reaches the beginning of the measured distance. The diaphragm dips into the water quietly without disturbing the regimen of the stream. The time of transit over this distance is observed, from which the velocity of the diaphragm can be computed. The mean velocity of the water is usually assumed to be the same as the velocity of the diaphragm. A correction should, however, be made for the frictional resistance of the car, and for the velocities in the clearance between the diaphragm and the periphery of the canal, which are not integrated by the diaphragm. In well designed apparatus this frictional resistance and the clearance are so small (usually about ½ in.) that no serious error is caused by neglecting their effects, especially since they have a tendency to counteract each other on account of the smaller velocities occurring near the periphery. To obtain the discharge the cross-sectional area of the water must be known; the depth of water in the channel is, therefore, constantly observed during a gaging.

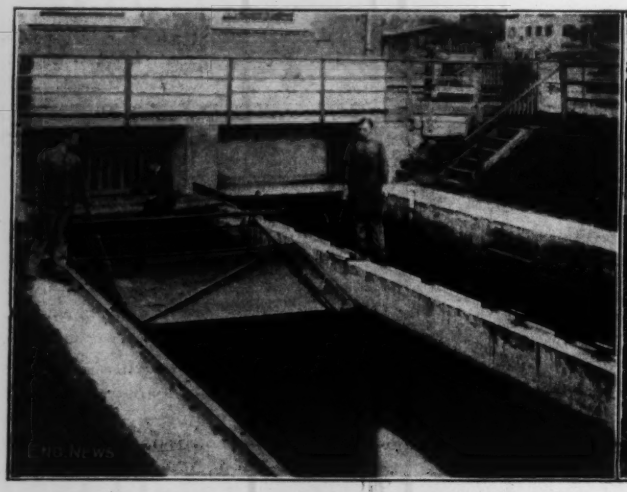


Fig. 1. Diaphragm before Dipping

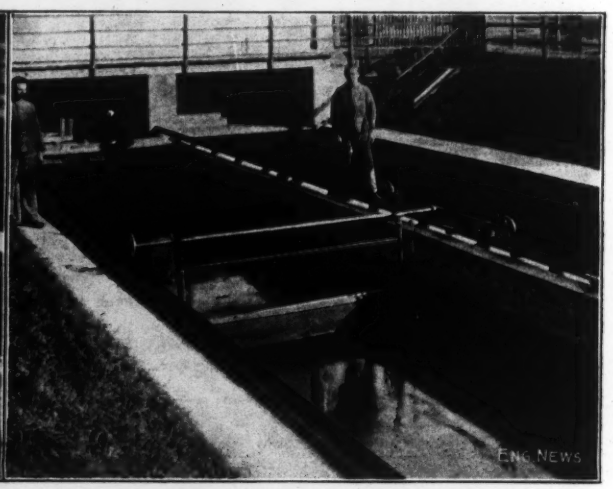


Fig. 2. Position of Diaphragm during a Gaging

(Rating Canal and Diaphragm in Heidenheim, Germany.)

The chief advantage in this method is the rapidity with which the measurement can be made. This is of importance especially in the testing of turbines, where it is rather difficult to keep operating conditions constant for a time sufficient to obtain a good current-meter measurement. In testing turbines installed at power stations, weir measurements are often impracticable on account of sacrificing part of the available head, and current-meter gagings, although the least expensive, require skilled observers, an accurate rating of the instrument and considerable time in computing the discharge. A complete diaphragm gaging can be made in a few minutes and the result computed immediately.

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†This scheme of measurement was employed in testing the turbines at the new municipal hydro-electric station at Winnipeg, Can. ("Eng. News," July 4, 1912, p. 12-20). The Canadian diaphragm apparatus differed from that described by Mr. Weidner, in that it floated on the water instead of being supported by wheel-trucks on rails.

Fig. 3 shows the schematic arrangement of an electrical recording device used in conjunction with the diaphragm at the J. M. Voith station at Heidenheim. K_1, K_2, K_3 are three electromagnets; pens L_1, L_2, L_3 are attached to the armatures of these in such a way that a break in the line traced on a moving chart, occurs whenever the circuit is closed. Next to the rail A, on which the diaphragm car runs, an electric circuit P has been arranged with 11 contact points spaced 1 meter apart. The battery N_1 and switch O_1 are connected in series with the

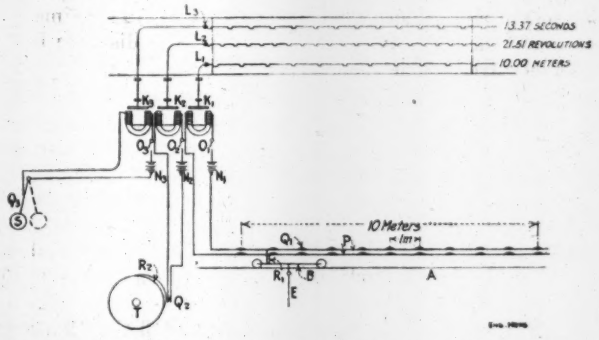

FIG. 3. WIRING DIAGRAM OF RECORDING APPARATUS AT HEIDENHEIM

electro-magnet K_1. When the shoe R_1 on the car B passes over the contact points Q_1, the circuit is closed; this fact is shown on the paper by a break in the line traced by the pen L_1. A similar arrangement is used to record the number of revolutions of the turbine. The eccentric R_2 on the turbine shaft T closes the circuit Q_2, and the pen L_2 records a break in the line for each revolution. The third pen L_3 is connected to the time circuit, every half-second being recorded on the chart. With a similar recording device used at the Berlin Technische Hochschule, two additional pens actuated by floats record the elevations of the water in the head- and tail-races.

The discharge at the Voith testing station was formerly measured by current meters, as the head was too small to permit the use of a weir. The current-meter gaging took from ½ to ¾ hour and if one considers that several measurements of the discharge must be made for a complete turbine test, it may readily be seen what a saving in time the diaphragm method accomplishes.

The disadvantages of the diaphragm method are that a channel of sufficient length and uniform cross-section must be available, and that the cost of installing the necessary apparatus is rather high. A diaphragm and its appurtenances installed at the hydro-electric plant at Ackersand, Switzerland, cost approximately $580, which includes the recording apparatus. For these reasons, the method is limited to the measurement of moderately large quantities of water, and its application will probably be restricted to hydraulic laboratories, turbine-testing stations, and places where the apparatus can be used often and first cost is, therefore, not such an important factor. For high-head power plants or small low-head plants, the installation of the diaphragm apparatus would be of considerable commercial importance, inasmuch as the operating efficiency of the turbines could be obtained at all times with a few simple measurements.

The new method could also be applied to discharge measurements in the main canals of irrigation systems, for the successful operation of which the measurement of the water is essential. It is not likely that any device can supersede the weir, for the measurement of the flow in small laterals; but for the larger canals the diaphragm method possesses advantages which in some cases would make its use preferable to either the weir or current meter. In many channels there is not sufficient fall to permit the use of a weir, and in others the silt-bearing streams affect its accuracy. Rating flumes could easily be calibrated and a recording device installed to give a record of the total flow.

Professor Andersson has used the method extensively in the Scandinavian countries for testing turbines at power plants; and in places where a suitable canal was not available a wooden flume was built. Wherever possible, a length of from 50 to 100 ft. was chosen for the rating flume, but in some cases it was possible to get a length of only 33 ft. The actual gaging distance was then only from 10 to 13 ft. Where the car and track were not available, the diaphragm was guided by wooden strips in the passage over the gaging distance. Some of the canal sections were quite large, as quantities as high as 830 cu.ft. per sec. have been measured.

Diaphragm measurements are subject to the following errors: (1) in observing the time of transit; (2) in observing the depth of water; (3) in estimating or neglecting the frictional resistance of the car; (4) the velocities in the clearance; (5) the wind pressure on that part of the apparatus not submerged; (6) the possibility of the diaphragm rubbing against the periphery of the canal;

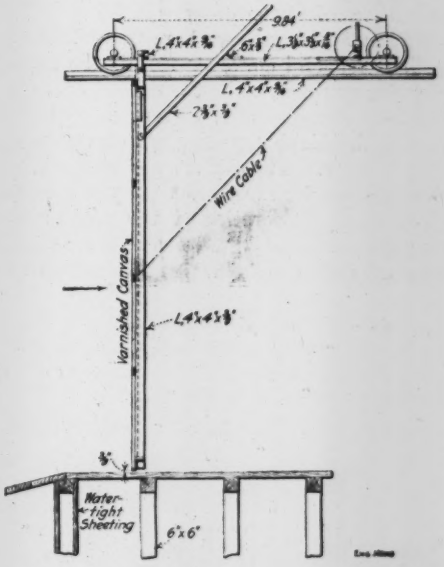

FIG. 4. CROSS-SECTION OF DIAPHRAGM AND CAR USED AT NOTODDEN, NORWAY

(7) foreign matter becoming wedged between the diaphragm and periphery.

As previously mentioned an extensive series of experiments* was made by the Swiss Bureau of Hydrography to determine the accuracy of the diaphragm method by

*The details of these experiments will be published shortly in a bulletin of the University of Wisconsin, abstracted from "Vergleichs-Versuche mit Flügel und Schirm-Apparat zur Bestimmung von Wassermengen," by Otto Lütschg, Adjunkt der Schweizerischen Landeshydrographie, Bern, Switzerland. Price, 1.50 francs.

comparison with weir and current-meter measurements. The results of these experiments may be summarized in the following conclusions:

(1) The diaphragm gagings checked the meter gagings in all cases within 1%; the discrepancies being in some cases positive and in others negative. (2) The diaphragm gagings checked the weir gagings in all cases within 3%. (3) The rating canal for diaphragm gagings should be of sufficient length to overcome the effect of pulsations in the stream. (4) The diaphragm should be placed in a vertical position in making a gaging. (5) The diaphragm method of gaging will give equally good results with perturbed flow as with parallel flow.

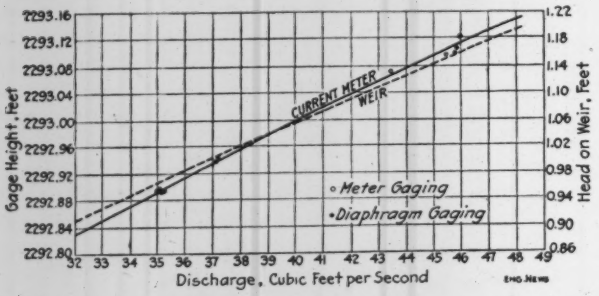

FIG. 5. COMPARISON OF SIMULTANEOUS DIAPHRAGM AND CURRENT-METER GAGINGS

In these experiments the current-meter gagings were considered more accurate than the computed weir gagings, because the weir was not of standard form and it was difficult to select the proper coefficients. The results of the different gagings have been plotted and are shown in Fig. 5. The solid curve represents the average of the plotted meter gagings; while the dashed curve is the computed weir-discharge curve. From this diagram it can be seen that the diaphragm gagings check the meter gagings very closely. The differences range between such small limits, that for most practical purposes they are of no significance. Theoretically, the results of the diaphragm gagings should be too small rather than too large, as the velocity of the diaphragm is retarded by the frictional resistance of the car, for which no correction was made. It seems, however, that the effect of this resistance is offset by the smaller velocities existing in the clearance between the diaphragm and the periphery of the canal, which are not measured by the diaphragm.

The application of the diaphragm is obviously limited; but no one method will apply to all situations. Past experience with the diaphragm in Europe has been very satisfactory; and it will undoubtedly come into use in this country, where suitable canals are available and the cost of installation is not too great. In view of its advantages and accuracy, the diaphragm method deserves recognition as a valuable contribution to the science of hydrometry.

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Rigid Overhead Conductors for Electric Traction, Michigan Central Terminal, Detroit

The Michigan Central Tunnel under the Detroit River at Detroit, built in 1910, is equipped for electric traction with bottom-contact third-rail identical in form and arrangement with that used in the New York Central terminal at New York City. The same equipment was extended into the new terminal yard and station about a mile west of the tunnel, completed a few months ago. However, some special requirements in the east approach to the station called for various short sections of overhead conductor, and in the trainshed of the station overhead conductor is used exclusively. The conductor-rail potential is 660 volts.

A rigid-conductor overhead system was designed for these uses. It is working satisfactorily, and while relatively high in cost it appears to be an excellent solution of the problems of the case.

RIGID SUPPORTS—In the trainshed, which is of the low-roof type, the rigid overhead conductor-rail is supported by hangers bolted direct to the roof framing of the trainshed. In the yard, transverse bridge supports were built at intervals of about 100 ft., spanning one, two or three tracks as required. The bridges are similar to signal bridges, or to the bridges used for cable overhead work in such systems as that of the New York, New Haven & Hartford electric zone (New York, N. Y., to New Haven, Conn.). The bridges support longitudinal trusses, one over each track, and to the bottom chords of these trusses are attached transverse beams to which the conductor-rail supports are bolted.

The relative arrangement of track and bridges is indicated by the sketch, Fig. 1, showing in plan a part of the east station approach trackage.

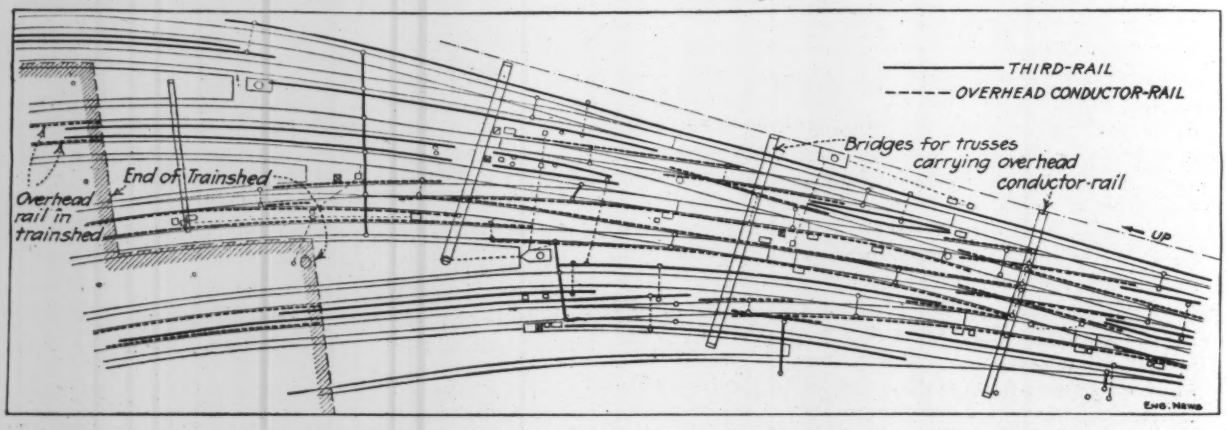


FIG. 1. PART OF UP-GRADE TRACKS IN EAST APPROACH TO MICHIGAN CENTRAL TERMINAL, DETROIT

CONDUCTOR-RAIL AND HANGERS—While the third-rail (at track level) is a 70-lb. bullhead rail, the overhead conductor is a 30-lb. T-rail of standard rail section, though of special composition. It is held in two-piece porcelain insulators engaged by hanger straps which fit in T-notches in the sides of the insulator. The details above the insulators vary with the nature and position of the supporting structure. Several typical hanger constructions are grouped in Fig. 2.

CONDITIONS REQUIRING OVERHEAD CONDUCTOR—The crossovers in the approach tracks, though laid out for No. 8 frogs, involve a gap between facing ends of third-rail of as much as 90 ft. On the westbound or upgrade tracks, where the gradient is 1% and over, so long a period of no current-supply to a train (using a single locomotive-unit) would produce enough retardation of the train to bring about harmful jerking when the shoe makes contact again. On this account, it was considered essential to provide overhead conductors for bridging the gaps. No such conductors are provided on the eastbound or downgrade tracks, or on the west side of the station, where the gradient is only 0.23%.

In the trainshed, the use of third-rail (at ground) was contemplated at first. This was on the basis of continuous unbroken tracks through the shed. But later, for convenience in handling trains, crossovers were laid between the pairs of tracks at midlength of the trainshed. These introduce the same considerations in the matter of long gaps in the power-supply as prevail in the upgrade tracks of the east approach; the gradient at the east end of the trainshed is 1%, and a large part of a train would still be on the heavy grade (hence requiring heavy power) while the locomotive was on the crossover. Therefore, the addition of the crossovers led to providing overhead conductor at the crossovers, and for simplicity and to eliminate any chance of men or tools getting in contact with conductor rails in the care and inspection of cars and interlocking equipment, the overhead system was adopted for the entire length of all the trainshed tracks, eleven in number.

J. C. Mock, Electrical Engineer of the Detroit River Tunnel Co., was in charge of the design of this equipment, under George H. Webb, Chief Engineer.

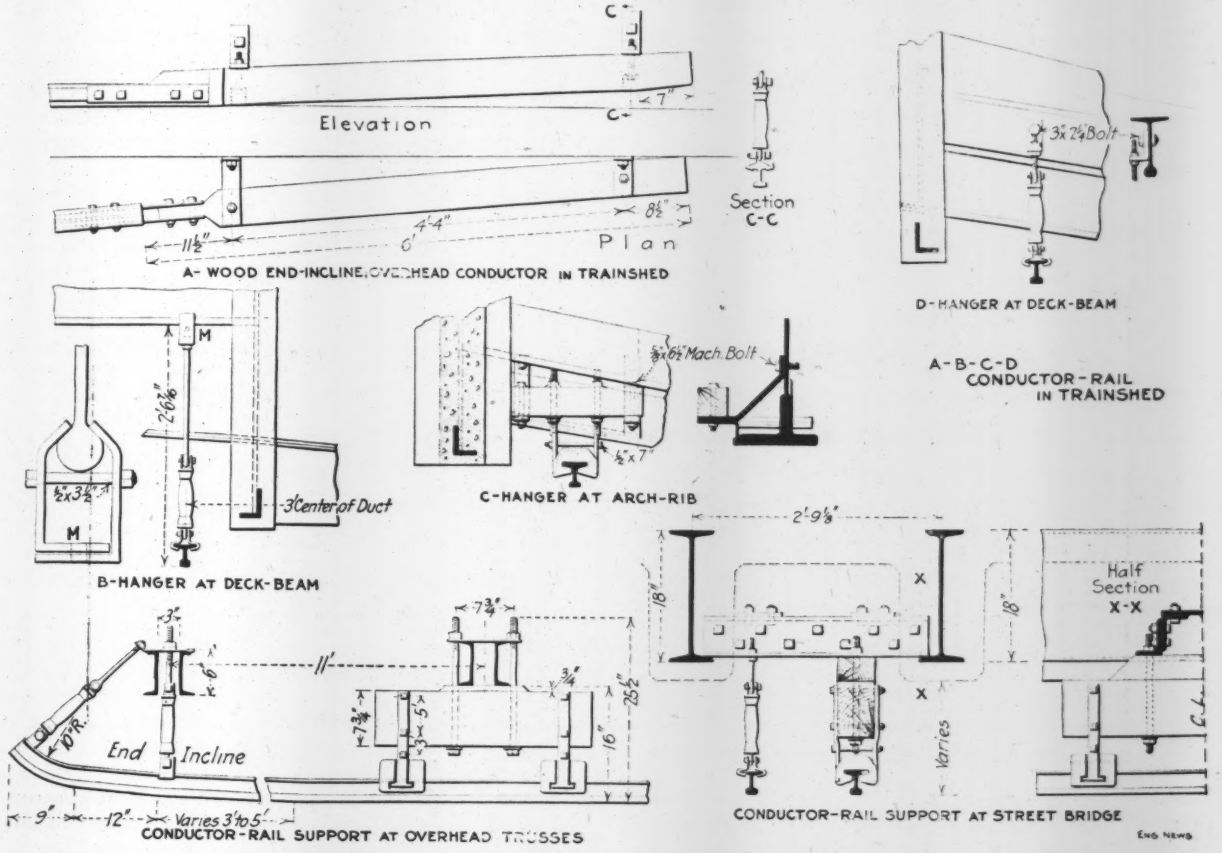


FIG. 2. OVERHEAD CONDUCTOR-RAIL HANGERS IN APPROACH AND TRAINSHED OF MICHIGAN CENTRAL TERMINAL, DETROIT

Paving for City Street Pavements—An investigation of the methods in vogue in 50 large American cities of paying for pavements has been made by a committee of the Cleveland, Ohio, Chamber of Commerce. The results are as follows:

New Paving—

In 62% of these cities the property pays all.

In 6% the property pays 50%.

In 10% the property pays more than 50% and less than 100% except in one city where the property pays 30%.

In 22% of these cities the city pays all.

Repaving—

In 42% of these cities the property pays all.

In 10% the property pays 50%.

In 8% the property pays between 50% and 100%.

In 40% the city pays all.

Approximately 25% of the cities pay the major part of the original paving, while 40% of the cities do the major part of repaving.

The Seros Hydro-electric Development near Lerida, Spain

REGULATING RESERVOIRS, EARTH DAMS AND POWER HOUSE

BY J. C. STEVENS*

SYNOPSIS—The diversion dam, canals and gates of the notable hydro-electric development at Seros, near Barcelona, Spain, were described in the previous issue. The present article is complementary, describing the seven earth dams which create the four regulating reservoirs, together with the power house.

REGULATING RESERVOIRS

The water withdrawn from the River Segre near Lerida for the hydro-electric plant near Seros is led through some 17 miles of canals, the construction of which has already been described. About three miles before reaching the power house, the canal line crosses a series of low valleys, seven of which were closed by earth dams to creat four regulating reservoirs. The arrangement of these is shown in the general plan (Fig. 1).

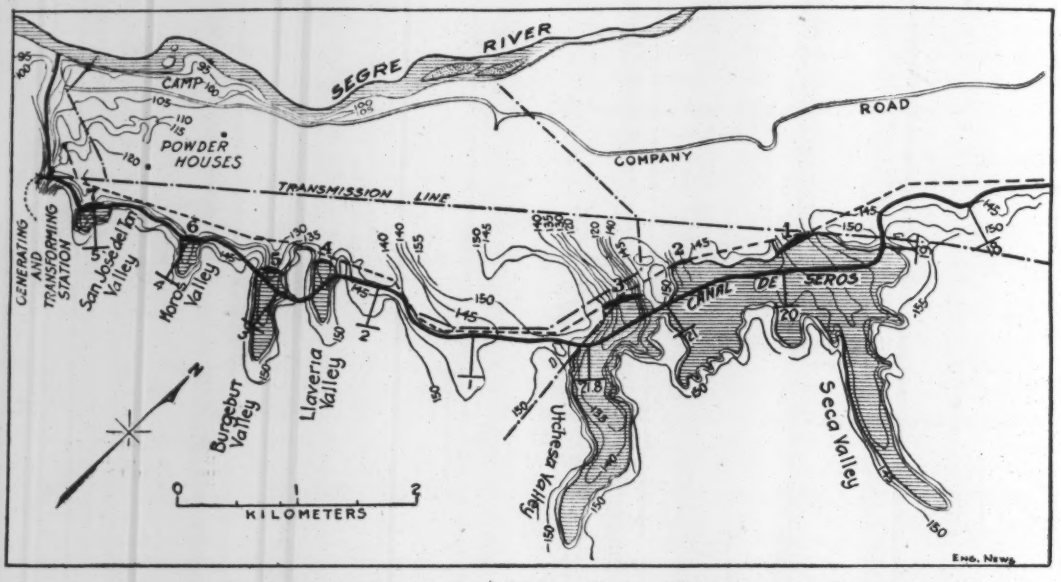


FIG. 1. REGULATING RESERVOIRS, LOWER CANAL, STATION AND TAILRACE

DAMS

The following table gives general quantitive data for each of the earth dams:

Dam No.	Maximum height, ft.	Maximum width of base, ft.	Length of crest, ft.	Cu.yd. embankment	Sq.yd. rip-rap
1	57	312	1273	270,400	19,000
2	41	223	836	58,890	6,400
3	75	436	1312	497,250	26,400
4	33	104	510	41,508	5,230
5	40	213	632	75,634	8,000
6	30	164	351	42,510	4,230
7	47	236	781	59,890	2,270
Total				1,046,292	71,530

In Fig. 2 is shown plan and cross section of Dam 3, the largest of the group. The same general design is followed for the remaining six.

The embankment for Dams 1, 2 and 3 were made largely from borrow pits excavated by steam shovel and by standard graders, the former working in borrow pits on the hill sides above water line of the reservoir and the latter in the beds of the "barrancos" upstream from the dams. The remaining dams were made by slips, wheel scrapers, carts or push cars in a manner to best suit the local conditions of each. About one-half of the total earth embankments were taken from the canal section. Steam rollers were used on Dams 1, 2 and 3, but on the others a smaller rollers, drawn by mules, was used. This roller was made of a series of cast-iron disks or wheels, with faces about 1½ in. wide. They were fixed on parallel axles and interlocked to keep the wheels from clogging with mud. The bed could be loaded with stone to any desired weight.

The cut-off trench on Dams 1 and 2 was excavated about 4 m. deep in the bed of the valley, and then a row of steel-sheet piling was driven in the bottom of the trench to refusal. Some of these penetrated to a depth of 12 m. The trench was then back-filled with puddle, inclosing the upper end of the steel sheeting.

On the cañon sides, the cut-off trenches were refilled with puddle as the dams were raised. Where the trenches cut the horizontal rock strata of the cañon sides, keys were built into the rock that also projected well into the puddle, forming a sort of cut-off collar to break the seepage plane along the faces of the rock strata. These keys are shown in Fig. 2.

The foundations of all dams were well stripped of all vegetable matter and the portion under the "B" section

*Spalding Bldg., Portland Ore. (formerly Project Engineer Ebro Irrigation & Power Co., Ltd., Barcelona, Spain).

was excavated 1½ ft. minimum, to insure a good bond with the original ground.

Outlet pipes were laid in trenches excavated in the original ground at the sides of the cañons under Dams 1, 2 and 3. These pipes were 1.0 m. diameter, made of 6 mm. steel plate, riveted in 6-m. lengths, and joined by heavy cast-iron flanges bolted together. The trench was then refilled with concrete, leaving a minimum thickness of 20 cm. around the pipe. A large cut-off collar surrounds the pipe at the point where it crosses the puddle trench, and smaller collars project from the concrete at each joint of the pipe. Above the concrete, the trench was refilled with clay and gravel puddle, which joined with the earth embankment of the dam.

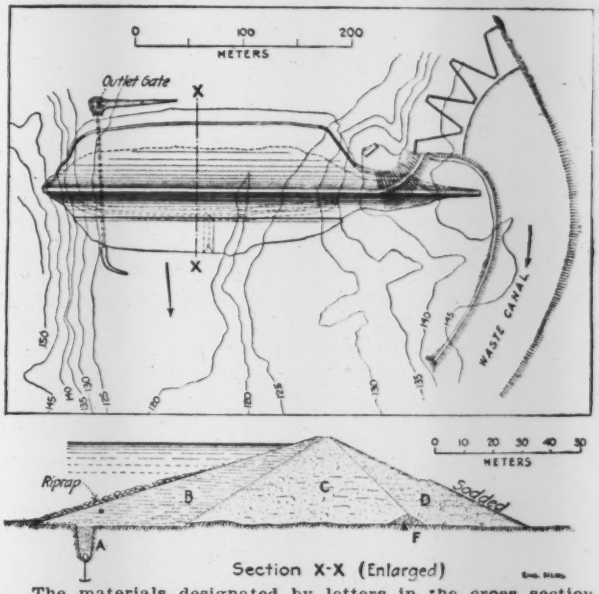
At the upper end of the pipes is a reducer section and a gate valve, 80 cm. diameter, operated by a torsion shaft with the mechanism on top of a tower frame.

The first reservoir is formed by Dams 1, 2 and 3. The connection between Dams 2 and 3 is an unlined channel 40 m. (131.2 ft.) wide on the base, with side slopes roughly ½ on 1, the bottom being level. The second reservoir is formed by Dams 4 and 5, the connection between them being an unlined canal with bottom width 18.0 m. (59.03 ft.) and side slopes 1 on 1. The third reservoir is formed by Dam 6 and the fourth by Dam 7. No spillways are provided for the second, third and fourth reservoirs, but a spillway at the left end of Dam 3 (Fig. 2) will provide a discharge capacity of 296 cu.m. per sec. (10,000 sec.ft.) under 0.65 m. (2.25 ft.) head. This is of the "zigzag" type, 304 m. (997 ft.) long, and operates entirely on freeboard. A discharge channel with 300 cu.m. per sec. capacity discharges the water into the cañon below the dam. This spillway will not come into action except in the case of an unprecedented flood on the drainage area above the dam, coincident with the stoppage of the power canal. The power canal has sufficient capacity for all known floods.

Filling Reservoirs and Testing Dams

The filling of the reservoirs was attended with some settlement, and consequent cracking of the earth dams. The seepage for each was carefully collected and measured over Cippoletti weirs, readings being taken every two hours. The total seepage from all of the seven dams did not exceed 3.0 sec.ft. with reservoir full. The seepage increased uniformly with the head, but whenever the head remained stationary, there was noticed a tendency to decrease.

Little or no seepage occurred through the dams themselves, but nearly all of it occurred in the horizontal strata of porous rocks that are exposed in the side of the cañons. Seepage through these original hillsides could not be prevented. In some cases the seepage appeared below the dam, in some it came to the surface within the section of the dam near the back side. Where this occurred, deep trenches were dug into the hillside, following the toe of the embankment, with spurs where necessary to intercept the water. These trenches gave an easier outlet for the water and prevented saturation of the earth embankment. They were refilled with large, broken stone of a uniform size to prevent caving. The rock dike, Fig. 2, answered its purpose of a blind drain, and in nearly all cases water began to weep from them after the reservoirs were filled. The greatest seepage occurred from Dam 1. A spring of crystal-clear water developed from the outlet of the rock dike, that runs about 80 gal. per min. with the reservoir full. The water undoubtedly came from the cañon side and followed the rock dike to its outlet at the lowest point of the valley. The remainder of the seepage came from the horizontal rock strata in the cañon sides except a small portion that appears as ground seepage in the valley below



The materials designated by letters in the cross section are:

CLASS A—Puddle refill of the cutoff trench; equal parts of clay and bank gravel in which all stones over 1 in. diameter were rejected. Clay and gravel hand mixed at the side of the trench and shoveled into it, thoroughly mixed with water and compacted by tampers.

CLASS B—Natural bank clay, free from stones larger than 2 in. diameter, spread in 8-in. layers inclined at about 3% towards the upstream face of the dam, sprinkled and rolled.

CLASS C—Borrow run earth, or clay and gravel of coarser material than Class B. In placing the finer and more compact material was placed in the upstream portion. To accomplish this the "C" section was divided into four equal parts and the limiting sizes and quantity of stones were fixed in each quarter in such a manner as to secure as far as practicable a gradual decrease in the imperviousness from the front to back faces of this section without sacrifice of density.

CLASS D—Borrow run of coarse material on earth and gravel with larger stones intermingled and spread in layers of about 30 in. with only sufficient rehandling to insure a dense mass and at the same time pervious to water filtration.

CLASS F—A rock dike built transversely across the dam near the lower face to act as a blind drain to relieve any filtration that may penetrate the face of the dams.

Fig. 2. Earth Dam No. 3, Seros Project

the dam. A trench was dug along the entire length of the toe of this dam, and all saturation of the body of the dam effectively prevented in this manner. The total seepage from the dam is 30 liters per sec. (1.05 sec.-ft.) with reservoir full.

Transverse settlement cracks developed along the shoulders of the embankment on all dams. These cracks continued to widen from a hair crack to about 1½ in. wide in some cases, and then appeared to remain stationary. They were sealed by drilling 3-in. holes along the crack and refilling with a liquid clay puddle.

Dam No. 3, which will take its place among the high earth dams of the world, showed very little settlement and comparatively little seepage. The total seepage with reservoir full was 0.64 sec.-ft., all of it coming from the rock strata in the original hillsides.

It is next to impossible to build an earth dam that will not leak, and as long as the seepage water remains clear there is no apprehension. To have completed seven

earth dams on one project in which the combined leakage is less than 3 sec.-ft., or 1 part in 1400 of the flow from the reservoirs, is a source of no little satisfaction to the writer.

Generating Station

The power house is provided with four 16,000-hp. Francis-type turbines on vertical shafts, each direct connected to a 10,000-kw., 50-cycle, three-phase alternator generating at 6000 volts and revolving at 250 r.p.m. Room is left for a fifth unit of similar dimensions. The maximum head is 49.5 m. (162.5 ft.) and the minimum 46.0 m. (151 ft.). Excitation is secured by two 600-hp. 600-r.p.m. turbines direct connected to 300-kw. generators, or by a motor-generator set. Current is stepped up from 6000 to 110,000 volts in the transformer house, which is built as a portion of the main station.

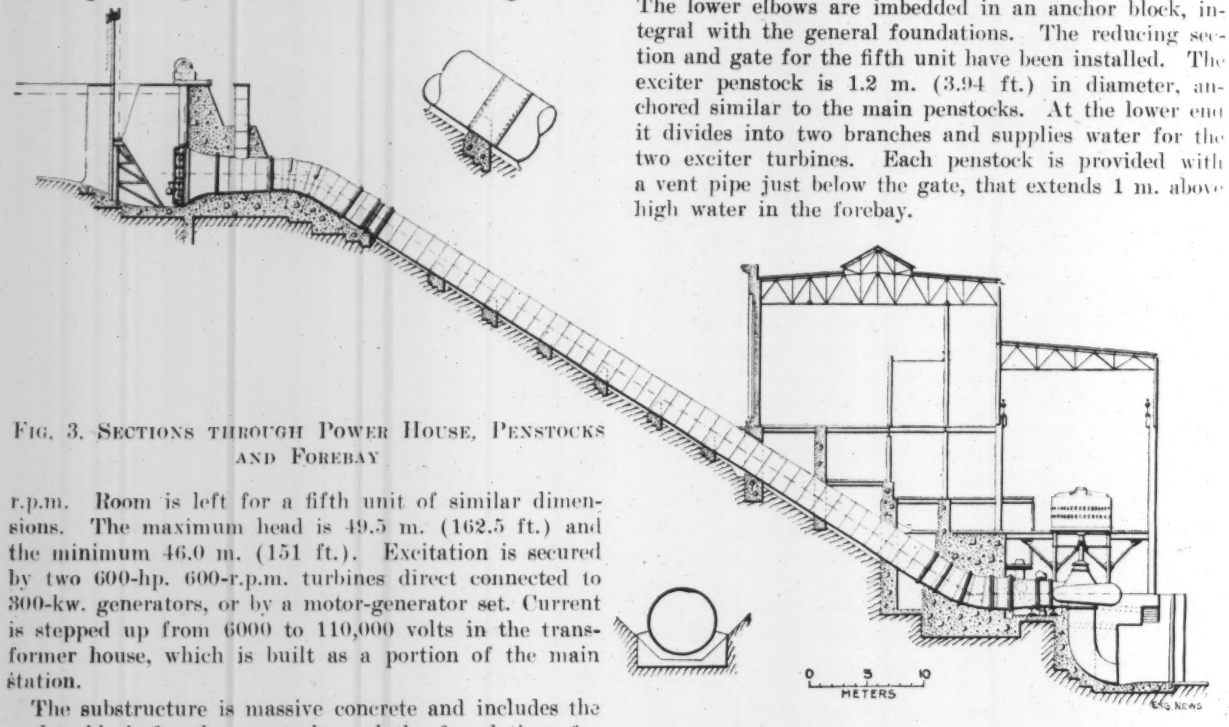


Fig. 3. Sections through Power House, Penstocks and Forebay

The substructure is massive concrete and includes the anchor block for the penstocks and the foundations for the turbines. The latter have their draft tubes formed in the solid concrete. The generator floor, walls, cranes, tracks, roof trusses and inside steel work are supported by structural steel columns. The walls and partitions of the superstructure are of local brick. The roof is a 3-in. concrete slab, laid on ribbed metal and waterproofed with asphaltum. Some of the floors were also of concrete slab with ribbed-metal and plain round steel-rod reinforcing, but the greater portion are of concrete supported by brick arches resting on the I-beam flanges of the floor system. The Spanish masons are very adept at building the brick arches, a single board cut to the span and rise desired sufficing them for centering forms. An 80-ton crane traverses the generator room and a 20-ton type runs over the transformers. There is the usual machine-shop equipment.

The penstocks for the four main units are of 10-cm. (⅜-in.) steel throughout, 3.0 m. (9.84 ft.) inside diameter, equipped with expansion joints, just beyond the upper elbows, which are anchored in the forebay wall. The lower elbows are imbedded in an anchor block, integral with the general foundations. The reducing section and gate for the fifth unit have been installed. The exciter penstock is 1.2 m. (3.94 ft.) in diameter, anchored similar to the main penstocks. At the lower end it divides into two branches and supplies water for the two exciter turbines. Each penstock is provided with a vent pipe just below the gate, that extends 1 m. above high water in the forebay.

The gates are 5.4x5.4 m. (17.71 ft.), with two small gates in each. The gates close by their own weight, and are opened by lifting chains without counterweights, operated by motor or by hand. The small gates are for filling the penstocks before opening the main gates, and operate automatically. At the lower end of each penstock is a butterfly valve, operated by oil pressure.

Before entering the penstocks, the water passes a set of screens with bars spaced 25 mm. These screens are in duplicate, each being suspended independently by lifting chains running over a chain block, by means of which one screen may be removed for cleaning while the other is in service.

The entire project was built very quickly. Active work began in January, 1913. Water reached the forebay on Mar. 10, 1914, or 15 months thereafter. The construction of the project involved the quantities shown in the following table:

Right of way	717 hect. = 1800 acres
Excavation exclusive of borrow for dams	1,951,900 cu.m. = 2,540,000 cu.yd
Embankment in dams	813,730 cu.m. = 1,065,000 cu.yd.
Structural steel	1909 metric tons = 2110 tons
Concrete and masonry	128,450 cu.m. = 167,800 cu.yd.

Turbines and penstocks were furnished by Escher Wyss & Co., of Zurich, Switzerland; the various gates and gate mechanisms were from Ransoms Rapier, Ltd., of London; racks and screens came from Ammie Geiscke & Konegan, of Zurich. The electrical equipment was made by the General Electric Co., of Schenectady, N. Y.

Fig. 4. Power House Exterior. Seros Project

U. S. A. Practically all of the steel was furnished by the Blaw Steel Construction Co., of Pittsburgh, Penn., U. S. A.

The work was carried out by the Ebro Irrigation & Power Co., Ltd., under the supervision of the Pearson Engineering Corportion, Ltd., of New York. The following are the officials in charge of the work: F. S. Pearson, President; A. W. K. Billings, Vice-President and Managing Director; Fred W. Abbot, Manager of Construction; J. A. Sargent, Chief Engineer. The work was executed under the inspection of the Public Works Department of Spain, Sr. Don José Bores, Chief Engineer for the Province of Lerida.

A Dangerous Piece of Subway Construction

Building the Boylston Street subway in Boston* past a poorly founded church tower, the Old South Church, cor. Dartmouth and Boylston St., was described a short time ago by L. B. Manley before the Boston Society of Civil Engineers, the work having been successfully completed. Prior to this, the operations at Old South Church have been withheld from publicity, the engineers of the Boston Transit Commission apparently having in mind the chance of a bad accident.

The stone tower adjoins the church and is continuous with its masonry; it is a rectangular prism, 25x28 ft., with turrets and a stubby spire, 231 ft. high above sidewalk. It stands within 20 ft. of the street line. The Copley Square Station of the subway comes right opposite. Although the northerly station platform is omitted along the church, and though the subway itself was crowded over to the opposite (south) side of Boylston St. as far as possible, the subway comes within 27 ft. of the tower foundation, and within about 35 ft. of the tower itself. The tower rests on a spread footing, 37x42½ ft., of granite, supported on piles.

The subsoil here is filled material for about 20 ft. below sidewalk level, resting on 90 to 100 ft. of silt and clay, with pockets of sand and gravel in its upper 16 ft. The piles on which the church tower rests were driven into this stratum and were cut off about 6 ft. above the bottom of the fill, or about 15 ft. below sidewalk level. The bottom of the subway structure is 30 ft. below sidewalk, or about 15 ft. below the base of the tower foundation masonry. The double-track subway structure and station integral with it are a reinforced-concrete structure designed strong enough to distribute its load uniformly over its base area, so as to avoid the possible difficulties of a pile foundation.

The church tower (with foundation) is estimated to weigh 5000 tons, giving a uniform base pressure of 3.8 tons per sq.ft. However, shortly after the completion of the tower in 1875, it started to lean toward the street, finally attaining a maximum lean in the direction of the corner of about 3 ft. This increases the pressure in the lower corner of the foundation. With a 30-lb. wind in the same direction, the greatest soil pressure under the tower at the leeward corner nearest the street would be over 5 tons per sq.ft. The number and length of the piles are not known, but in any event the pile loading is greatly excessive. Under these circumstances, there was good reason for fearing that the subway excavation, only 27 ft. away, and 15 ft. below the bottom of the masonry, might start a soil movement which would threaten disaster.

A reassuring feature is the fact that in this whole region there is direct evidence that the ground has subsided or shrunk vertically to an amount of from several inches up to 2 ft. The indications are that this is due to a compression and settling of the great depth of silt. If the leaning of the tower could be charged against this same effect, there would be no reason to assume the tower foundation to be weak.

The Boston Transit Commission prescribed in detail the method of construction to be followed at the Copley Square Station, and assumed the responsibility for the safety of the tower if its prescriptions were followed.

It was decided not to try to underpin the church tower, but to depend upon the old foundation and conduct the subway work in such a way as to prevent any possible movement of earth. The essence of the method was building the subway in successive transverse slices, 12 ft. long in the direction of the subway, each fully sheeted, one slice of the subway structure being completely built before the next slice ahead was excavated more than 10 ft. Steel sheeting around the slice was driven before excavation, and this sheeting in turn was stiffened and supported by constructing a "self-supporting" concrete back-wall against the inside of the sheeting in a narrow trench. The back-wall and the sheeting were always driven and built half a slice ahead, and the back-wall being reinforced with horizontal rods, it had support at its rear end against the finished subway structure and at its forward end against the 5-ft. section of trench in which it was built; in addition, the wall was fully braced across the width of the subway excavation. Immediately upon completing the general excavation of a slice, the floor was surfaced off with concrete, waterproofing placed, the permanent reinforced-concrete floor constructed and then the side walls, columns and roof beams placed, and concreting completed.

Extensive grouting of the subsoil was done in connection with these operations, both before beginning excavation and after completion of the subway. Preliminary to the general excavation, the westerly end of the work was inclosed by a cutoff wall of steel sheeting on the line of Dartmouth Street, and the portion west of this line (i.e., in front of the church) was grouted with cement; grout forced in through 2-in. pipes jetted into the clay every 5 to 10 ft. The air pressure used for grouting was generally 50 lb. (90 lb. proved too high).

After completion of the subway structure, the space back of the longitudinal line of sheeting nearest the church was also grouted as thoroughly as was possible, in order to consolidate the soil near the church tower foundation and fill any voids that might have been created.

The subway construction progressed quite rapidly under the circumstances; the average time for constructing one 12-ft. slice, after work was fairly started, was seven days. The dangerous section was completed by December, 1913.

The tower increased its lean by about 1 in. during the operation, measured at a point about two-thirds of the height of the tower from the street. A settlement of ½ in. of the lowest point was noted.

*See article "New Subway Construction in Boston," "Engineering News," Dec. 11, 1913, pp. 1190-1192.

Concrete Lining, Franklin Canal, Rio Grande Project

By L. M. Lawson*

By Act of Congress, the benefits under the Reclamation Act were extended to the State of Texas. The Franklin Canal, which has been in existence for a number of years, was purchased by the Reclamation Service, to be reconstructed and enlarged and become the main canal to serve the 45,000 irrigable acres in the El Paso Valley. On the completion of the Elephant Butte Storage Dam, the total area to be irrigated under the entire project will be 180,000 acres, of which 20,000 lies in the Republic of Mexico. The Franklin Canal has its intake just above a concrete diversion dam in the Rio Grande near the city of El Paso. From this point it follows at the base of a steep slope along the river for one mile, passes through the lower portion of the city of El Paso on Eighth St. and continues through the central portion of the valley, a distance of 30 miles. Its maximum capacity, when purchased by the Government, was 160 sec.-ft. and approximately 12,000 acres were in limited cultivation under the irregular supply from the normal river flow.

Concrete-Lined Canal, Rio Grande Irrigation Project, El Paso, Tex.
(On this stretch of the canal 162 lin.ft. of lining were placed in eight hours.)

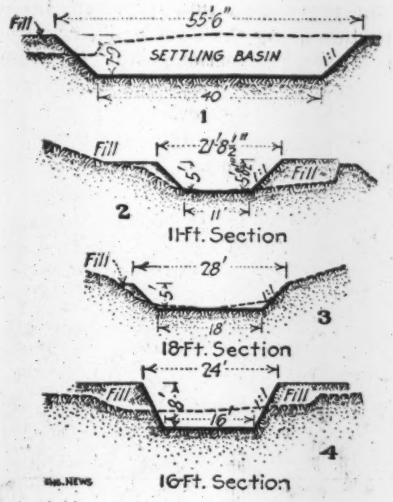
The intake structure, the diversion for sluicing, the bridge crossings and general condition of this canal were inadequate for the irrigation of any increased area and temporary structures would cost in maintenance, in a short time, more than permanent concrete structures.

The reconstruction plans proposed a new intake at the end of the diversion dam; a settling basin with sluice gates to discharge the heavy sand into the river; lining with concrete the portion along the river bank, and providing a rubble wall for river-protection; lining the 6000-ft. section through Eighth St., El Paso; the enlargement of the earth section below the city, and providing necessary concrete and timber and steel bridges and structures. With this enlargement it would be possible to divert 450 sec.-ft., permitting the area under cultivation to be increased from 12,000 acres to 45,000 acres.

The lining on Eighth St., which will be considered first in this article, was decided on as necessary in order to discharge the increased quantity within the right-of-way (25 ft.) granted by the city. The franchise provision, taken over with the canal when purchased, made it possible for the city authorities to enforce covering the canal through the city limits. To overcome this objectionable feature, a new franchise was submitted and passed, which specified certain bridge and fence work in lieu of the cover. The old earth-lined canal occupied 34 ft. of the 70-ft. street, and was crossed by five steam railways, two electric roads and three wooden street bridges. There were also 10 sewers crossing the canal location from 6 in. to 3 ft. above bottom grade.

The plan of cross-section adopted, permitted the greatest width of roadway on either side, and it is the intention of the city authorities to park and otherwise improve the remaining portion of the street. Soil and drainage conditions permitted a ½ to 1 side slope of the lined section on Eighth St., the 4 in. of concrete being placed with forms. In the settling-basin section, the side slopes were 1 to 1, with the concrete placed without forms. On account of the narrowing of the section (see Fig. 1, Sec. 4) and the low elevation of the banks of the old canal, forms

*Project Manager, Rio Grande Project, U. S. Reclamation Service, El Paso, Tex.



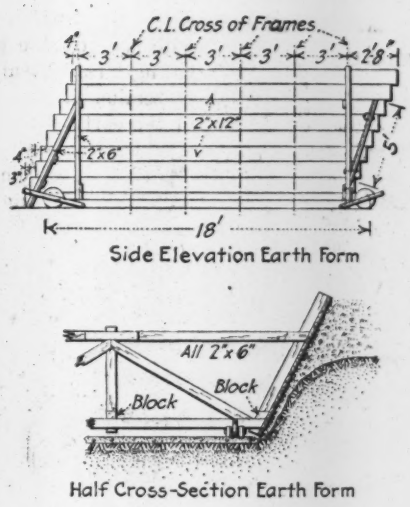


FIG. 1.

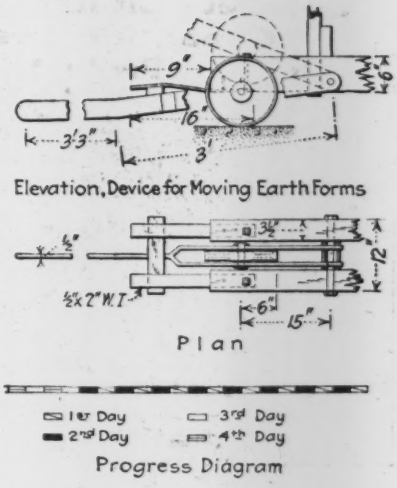


FIG. 2.

FIGS. 1-3. SECTIONS OF FRANKLIN CANAL, RIO GRANDE IRRIGATION PROJECT, AND DETAILS OF EARTH AND OF CONCRETE FORMS

were devised to permit the earth being tamped in place before concrete was poured. The procedure on the Eighth St. section was as follows: The bottom was excavated to the new grade, which in most cases was from 12 to 20 in. lower than the original grade. The sides were then trimmed roughly to line with slip scrapers, depositing the earth within easy reach to facilitate the back filling. The 4-in. concrete floor was then laid.

The earth form (Figs. 2 and 4) was then placed on the floor. This form weighed about three tons and was built to the outside lines of the concrete and could be easily lined up on the concrete floor. On each corner there was a steel lever and wheel. By lowering this lever the weight of the form was placed on wheels, which permitted the form to be rolled along on the concrete floor to the next position. When in the right position the levers were raised and the form allowed to rest on the concrete floor in position for back filling and tamping. The earth in most places was a light sandy loam, which, after being sprinkled and tamped, remained in place very satisfactorily (Fig. 5). After the form has been back filled to the required height, which in most cases was about 8 ft., the levers were lowered. This raised the form about 6 in., making enough side clearance so that it could be moved ahead without disturbing the tamped earth. There were three of these forms, one with each concrete gang. Some 12 to 14 men on the earth gang easily kept ahead of the concrete crew. The best progress in eight hours was 16 form lengths or 320 ft. The concrete forms (Fig. 3), which followed the earth forms were very similar in design, differing only in that the sides could be pulled toward the center before the form was raised. The equipment for each concrete gang was a ½-yd. mixer, 8 concrete buggies, one earth form and 6 concrete forms.

The alternate method of placing the sides was adopted. A runway over the forms was first used but this was afterwards abandoned as it strained the forms and the continual jar increased the tendency to float. The pouring took place on both sides at the same time. The earth bank

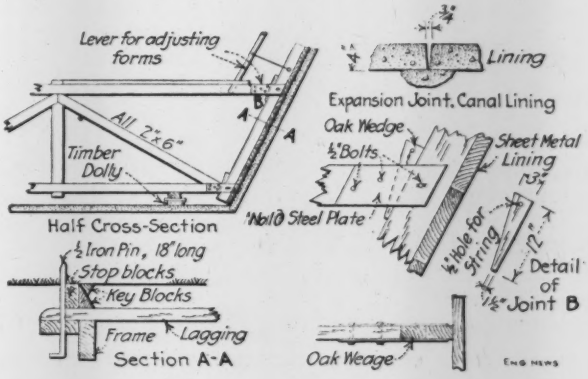


FIG. 3.

was sheltered by a piece of sheet iron and the concrete was brought up equally on each side, about 2 ft. at a time. The concrete used was a wet 1:3:5 mixture, without reinforcement. It was well tamped and spaded back of the forms.

Considerable difficulty was experienced in handling the forms at a number of bridge crossings and while all bridges were to be replaced with reinforced-concrete structures, several street crossings had to remain in a passable condition. On account of the short time in which to complete the work and furnish the spring-irrigation water, a large force of men was employed, working two shifts a greater part of the time. The limiting conditions on the work, where not hampered by bridges, sewer crossings and traffic restrictions, were the number of concrete forms per gang, as the ordinary concrete gang can handle 12 forms as well as six. One gang, where not hampered by the above obstructions, made nine 18-ft. sections in 8 hours with six 18-ft. forms.

In the settling-basin section the procedure was somewhat different. After excavation by teams, the floor was laid. The sides were then roughly trimmed, after which 2x4's were laid every 10 ft. (Fig. 6) along each side of the floor on a 1 to 1 slope. The concrete for the sides was mixed to a good pasty consistency, carried to the point of

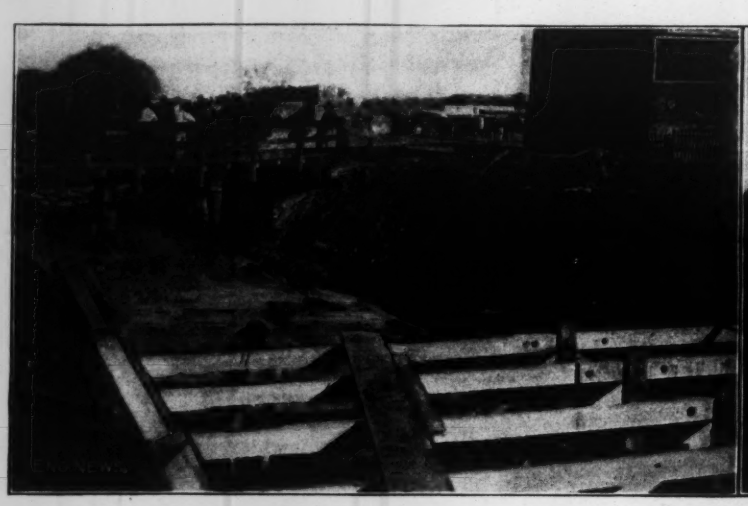


FIG. 4. FORM FOR EARTH BACK-FILLING GOING INTO POSITION

FIG. 5. EARTH SECTION OF BACK-FILLING COMPLETED, READY FOR CONCRETE

use in wheelbarrows and placed on the concrete slope with a square pointed, short-handled shovel and lightly tamped with the back of the shovel. An important point in placing the sides is to have the shovelers throw the concrete so that it lands in a compact mass with no tendency to spread, which gives a more even and better mass. It was interesting to note the addition of water to the mix as the men gained proficiency in the work. Beginning with a dry mix, they soon could handle it fairly wet. As soon as the 10-ft. space was filled, the 2x4's were removed and the space left was filled with concrete. The concrete as placed was left with a rough surface. A thin 1:3 mortar was made to plaster this surface and applied a few minutes after the concrete had been placed. Expansion joints were placed every 50 ft. throughout this section (see detail, part of Fig. 3).

The work on this division includes the lining of three different sections. Some of the best day's work on them were as follows:

	Sq.ft.
500 lin.ft. of floor 11 ft. wide	5500
286 lin.ft. of sides 7 ft. 11 in. high	
276 lin.ft. of sides 7 ft. 8 in. high	4180
370 lin.ft. of sides 11 ft. high (settling basin)	4071

The length of slope in the settling basin is 11 ft. and though this was placed with little difficulty it is about the limit in height for one course.

Instructions to begin this work were received on Dec.

FIG. 6. PLACING CONCRETE LINING WITHOUT FORMS; SETTLING BASIN OF FRANKLIN CANAL

20, 1913. Immediately material was ordered, men and teams secured and an organization assembled to handle this work in the three months available time. On Jan. 20, 1100 men and 700 animals were at work. The laborers were Mexicans receiving $1.25 per day; carpenters were, with a few exceptions, Americans and were paid $5 per day of eight hours. In general the skilled labor was sellected from various construction companies in El Paso. On Mar. 23, water was diverted into the completed canal and irrigation for the El Paso Valley began. Portions of

FIG. 7. CONCRETE-LINED CANAL ON 38° CURVE, EL PASO, TEX.

the finished structure are shown by the front-page view and by Fig. 7.

Tables I and II show the total quantities and unit

TABLE I. COST OF CONCRETE LINING ON EIGHTH ST., EL PASO (STA. 78-146)

(This lining was 4 in. thick at top, 5 in. at bottom. The canal had a bottom width of 16 ft., ½ to 1 side slopes, and a depth of 7 ft. 8 in. (Fig. 1); all side work was placed with special movable forms. A 1:3:5 mix was used.)

Total quantities: Cu.yd.	Lin.ft.	Sq.ft.	Total cost
2649.6	6066.7	203.835	$33,649.92

	Items	Unit costs: Cu.yd.	Lin.ft.	Sq.ft.
1.	Engineering, superintendence and inspection	$0.491	$0.199	$0.0063
2.	Form work, labor, material	2.120	0.925	0.0275
3.	Concrete, materials, mixing	5.290	2.310	0.0687
4.	Concrete, placing, finishing	1.650	0.720	0.0215
	Total	$9.551	$4.154	$0.1240
5.	Backfilling, trimming	3.150	1.370	0.0410
	Total	$12.201	$5.524	$0.1650

No. 3 includes cost of material, $3.54 per cu.yd., and all expenses incident to mixers and mixing, labor and repairs. The mixers made 25 separate moves and setups on this work.

No. 5 includes all handwork necessary before placing concrete but not the rough excavation to line and grade by teams. The original canal was 28 to 36 ft. wide, and about 3.5 ft. deep. The banks were raised 18 in. to 2 ft. and the remainder of the 8 ft. of depth was in excavation.

No general office expenses are included.

TABLE II. CONCRETE LINING STATIONS 0-38

(This includes three different water sections (see Fig. 1). No. 1, stations 0 to 6; No. 2, along river, stations 6 to 20; No. 3, along river, stations 20 to 38. All lining was 4 in. thick, laid up without forms on a 1 to 1 slope. The slope lengths on Sec. 1 were 11 ft., on Sec. 2 and 3, 7 ft. 1 in. to 7 ft. 8 in.)

Total quantities: Cu.yd.	Sq.ft.	Total Cost
1546.8	129,324	$13,145.23

	Items	Unit costs: Cu.yd.	Sq.ft.
1	Engineering, superintendence, inspection	$0.491	$0.006
2	Forms, labor, materials	0.243	0.003
3	Concrete, materials, labor	5.290	0.066
4	Concrete, placing, finishing	1.062	0.013
	Total	$7.086	$0.088
5	Backfilling, trimming	1.425	0.017
	Total	$8.511	$0.105

For conditions relating to Nos. 3 and 5, see footnotes to Table I.

costs of this work. The night work increased the cost probably 10% and with a less rush job a material reduction in all items is possible. All work was carried on by force account under eight-hour-day law. Otis L. McIntyre was Assistant Engineer in direct charge of the work, assisted by F. W. Cater, Assistant Engineer.

Old Wood Sewers in Boston, Mass.

(CONTRIBUTED)

The sudden sinking of small sections of the surface of some of the downtown streets in Boston, Mass., caused by the collapse of old wood sewers, has created considerable interest in them locally from time to time and has led to speculation as to the probability of serious damage being caused by such subsidence.

From an investigation by Edgar S. Dorr, of the Sewer Division, Public Works Department, it appears that in 1909 there were over ten miles of wood sewers in Boston. Many of them had been paralleled by masonry sewers or had been abandoned for some other reason, but it is estimated that even now there are in use not less than six miles of old wood sewers, which ought to be rebuilt very soon and that the cost of replacing them with permanent materials would be about $600,000.

These wood sewers were built many years ago, long before any comprehensive system of sewerage with intercepters and pumping stations was thought of, and as they were largely in low or filled lands and below the level of high tide, they were well soaked twice a day and consequently were not liable to decay as wood was cheap at that time, such sewers were economical to build. Under the conditions then existing the use of wood for sewers was good engineering.

In 1884, the growth of the city and the pressing need of discharging the sewage at more remote points led to the construction of an intercepting system from which the sea water was excluded by means of tide gates. This made a radical and unfavorable change in the condition of the wood sewers which, instead of being thoroughly wet twice daily, were wet only when they were filled by a very heavy rainfall. Long periods usually intervened between wettings and the decay of the tops of the sewers, beginning then, has been going on ever since, until now they are in such a condition that breakdowns are becoming increasingly frequent. Although they have not yet caused any serious damage, it is apparent that before long it will become necessary to replace them with modern sewers.

The Popularity of Engineering is shown in many ways. At a meeting of the Connecticut Master Plumbers' Association, a member objected to the name plumbers and said they should be called "sanitary engineers."

Field and Office

A Cable Drag Scraper on Mississippi Levee Work

The firm of R. H. & G. A. McWilliams, of Chicago and Memphis, has recently perfected and has now under operation on the Mississippi, near Memphis, Tenn., a cable drag scraper which is proving very successful in the construction of levees. The machine consists of the familiar drag-scraper bucket hung on a cable between two towers, one a braced tower of considerable height and independent rigidity, located back of the levee to be made and the other, a single low A-frame with counterweighted dead-man to balance the cable, located about 500 ft. riverward. The cable is pulled by a donkey engine located at the high or shore tower. The bucket picks up the earth from the borrow pit between the towers and drags it along to the levee where it is dumped, the empty and full bucket sliding over the levee, serving to compact the fill.

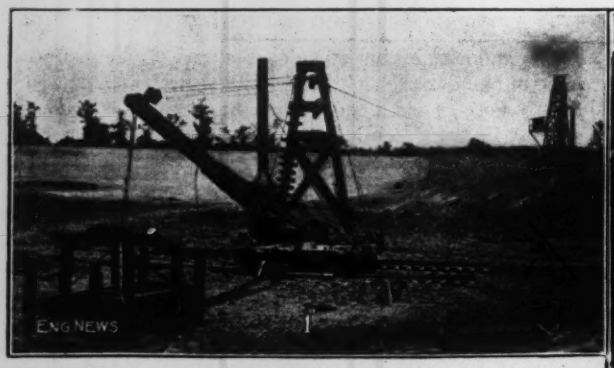


Fig. 1. Looking toward levee. Fig. 2. Looking toward tall tower.

A Cable Drag Scraper Building Levee at Scanlon's Landing, Mississippi River

The device serves admirably to fulfill the usual Mississippi levee specifications, which require that the levee be located from 1000 to 5000 ft. back from the river with earth for the construction taken from a shallow (3- to 8-ft.) borrow pit between the levee and the river.

The accompanying views show the machine in operation. Fig. 1 is a view from the low or river tower looking toward the levee under construction. The counterweight and trucks for moving the tower are clearly shown. Fig. 2 is taken looking shoreward and shows the bucket just after dumping and the tracks made by the bucket in scraping up the borrow pit. In Fig. 3, the bucket is just dumping on the levee.

The McWilliams outfit, now at use at Scanlon's Landing, is moving about 1200 cu.yd. per day at a saving of about 30% over the team-scraper methods usually employed. A 4½-cu.yd. bucket is used and it deposits layers of from 18 to 24 in. The round trip of the bucket averages about 2 minutes.

Excavating Railway Cuts with a Grading Machine

By J. R. Taft*

This method of taking out railway cuts is unusual, as local conditions in general do not permit the use of such an outfit except for work which may be spread over comparatively large areas at shallow depths, such as soil stripping, borrow pits, and the like.

The Halite & Northern R.R. is a standard-gage steam railway about 3½ mi. long, from Halite to near Retsof, Livingston County, N. Y. It is operated and maintained by the Genesee & Wyoming R.R., which furnishes connections with six trunk lines. The roadbed width at subgrade is 20 ft. in cuts (13 ft. 6 in. between ditches) and 16 ft. on fills, with slopes of 1 on 1½. The subgrade is about 18 in. below top of rail, with 7 in. of ballast under the ties. Fig. 1 gives the profile of the line.

Owing to the rolling character of the country, consequent long haul, and apparent absence of rock or stones, the contractors decided to take out the cuts by grading machine and wagons. The cuts so excavated contained altogether about 20,000 cu.yd., place measurement, or 57% of the yardage of the entire line, which averaged about 11,000 yd. per mile and aggregated about 35,000 cu.yd. Of the 20,000 cu.yd. in the cuts indicated, 94% was removed by machine. The remaining 6% was

*50 Church St., New York, N. Y.

unfavorable for machine work, being either root-bound surface soil or high places at the bottom of cuts.

An Austin elevating grader, hauled by a 20-ton steam tractor, was used for the work. The wagons were of 1½-yd. capacity, of the bottom-dump type, with three mules to each wagon. The outfit was supplemented with a light grading equipment of fresno, wheel and drag scrapers, for use in places inaccessible to the grading machine.

The material encountered was a stiff clay, hard and brittle when dry, but so plastic and slippery when wet as to be unworkable. Much ditching and draining were necessary. The work of grading was started about the middle of October, 1911. Owing to the effect of unfavorable weather upon the material, the grading machine was helpless during the greater part of November, December and January. Operations were suspended during February and March, 1912, and were resumed about the middle of April.

It was demonstrated early in the grading that a 20-ft. cut is too narrow for a machine to work in and load wagons when the cut has reached a depth too great for the wagons alongside to "straddle" the top edge of the slopes. This workable depth may be taken at about 3 ft. To secure additional width, the contractors were allowed to excavate as shown in Fig. 3. When the cut had

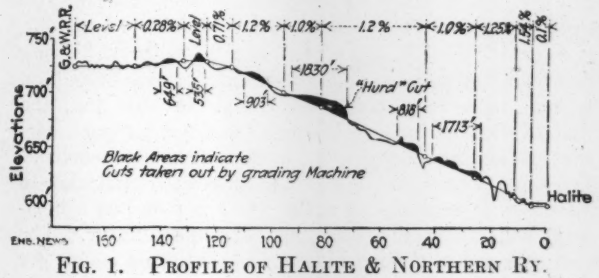


FIG. 1. PROFILE OF HALITE & NORTHERN RY.

reached a depth where the toes of the slopes were 30 ft. apart, the sides were taken down vertically to within 5 in. of subgrade.

The material shown in the hatched areas at the bottom was then cast into the double-hatched areas at the sides. In ordinary material this is not good practice. The results as a whole have proved generally satisfactory, and this method provided more room for working without enlarging the size of the cuts and thereby increasing the quantity of material to be moved.

Even a 30-ft. cut did not provide sufficient room for loading all the material direct to wagons working in the cut. There was plenty of room when excavating along either side of the cut and for about one-third the distance across the bottom from each side. In taking out the middle third, however, considerable rehandling was necessary. This meant that about one-third of the material was cast up on the sides, to be raked down by slopers into a position from which it could be again picked up by the machine. In order to do this, the machine was obliged to work casting up alone, which resulted in much "lost motion". This condition was relieved by putting the teams on light grading elsewhere while the machine worked casting up without the wagons, as shown in Fig. 2.

In material of this character, for about every 60 ft. the machine traveled, a wagon load of a little less than 1 cu.yd. place measurement, would be passed over the

FIG. 2. EXCAVATING A CUT WITH A GRADING MACHINE
(The machine is casting the dirt on the slope, to be shoveled down and loaded later.)

belt conveyor. Thus about 15 yd. were cast up for rehandling for each furrow so made through a cut 1000 ft. long. Therefore, in taking out the middle portion of the cut (six lifts of about 12 furrows each) the quantity of material turned over for a second handling runs into considerable yardage.

When the output of an organization is dependent upon a single one of its units, the results of its operation are measured by the efficiency of that unit. Therefore, to secure the most satisfactory results the unit must be worked to its economical capacity. This is not so difficult of accomplishment when operating under favorable conditions. On this work, however, such conditions did not exist. Bad water kept the boiler of the traction engine in a choked and leaky condition. Owing to this and the long water haul to reach some of the cuts, an extra tank wagon was required part of the time.

Stones embedded in the tough material when caught by the plow point threw severe strains into the whole apparatus, causing some of the most serious breakdowns. Such strains were relieved, however, by the shearing of the bolt in the plow frame which took the force of the impact. Hundreds of these bolts were broken and replaced, causing unrecorded delays of 5 to 15 min. each. The cramped area while working in certain cuts made it impossible at times to water the boiler from the tank wagons while in motion, which resulted in further delays at the turns. Owing to street paving and other activities in the cities, together with highway construction in near-by towns, it was extremely difficult to procure men and teams at a reasonable figure. Farmers were unwilling to subject their horses to the hard work of moving material of the nature encountered.

The total time of 119 days in which the machine was working in the cuts, and which does not include the removal of about 4000 cu.yd. from a borrow pit under more favorable conditions, may be classified as follows:

	Days	Per cent.
Unnecessary delays:		
Lack of duplicate parts	6	5
Delayed by work elsewhere on the line	5	4
Total	11	9
Necessary delays:		
General repairs	6	5
Wet ground	43	36
Total	49	41
Total delays	60	50
Machine in operation	59	50
Total working time	119	100

It will be evident that the work as a whole was not executed under favorable conditions. Eliminating the item of wet ground from the total working time of 119 days, 76 days were consumed in excavating about 20,000 yd., or a daily average of about 260 yd. This is less than

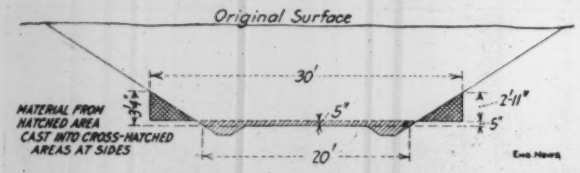


FIG. 3. EXCAVATING A 20-FT. CUT TO A TEMPORARY WIDTH OF 30-FT. TO ENABLE A GRADING MACHINE TO BE USED

half of what might be reasonably expected from a machine of this type working in similar material but in less constricted areas.

The contractor's daily expense for the operation and maintenance of this outfit may be considered as given in the accompanying table:

TABLE OF CONTRACTOR'S DAILY EXPENSE FOR GRADING

(A) At Working Point	
Foreman (member of firm)	$6.00
Tractor engineman	4.00
Tractor steersman	2.00
Machine operator	3.00
Tank-wagon driver	2.00
Extra tank wagon with driver	5.50
	$22.50
Fuel, oils, etc., for tractor	5.00
Six dump-wagon drivers, at $2	12.00
Two dumpmen, at $2.25	4.50
	$16.50
Total at working point	$44.00
(B) At Camp	
One blacksmith	$3.00
One barnman at $40 per month and board	2.00
One cook at $40 per month and board	2.00
	$7.00
Corral expenses for 25 head of mule stock	20.00
Total at camp	$27.00
Total of (A) and (B)	$71.00
For insurance, interest, depreciation, etc., 12½%	9.00
Grand total	$80.00

Assuming $80 as a fair figure for daily expenses, the cost of moving 260 cu.yd. per day is about 31c. per cu. yd. Board with lodging in camp was furnished the men at $4.50 per week, which was practically at cost. Corral expense is based on oats at 64c. per bushel, loose hay at $21 per ton, and straw at $10 per ton, all haulage by contractor.

With the exception of the barnman and cook, who were carried monthly, the contractor's force was employed on an hourly basis. In the event of extensive repairs or delays from various causes, the men were either put on other work or temporarily laid off. Although changing during working hours from one type of work to another does not lead to economical results, it sometimes avoids a total loss.

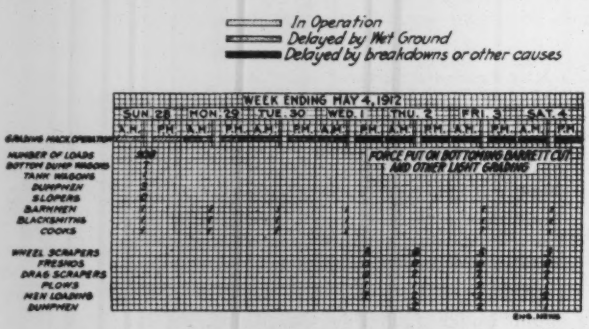


FIG. 4. TYPE OF CONTRACTOR'S WEEKLY LABOR REPORT ON RAILWAY EXCAVATION WITH AN ELEVATING GRADER

The writer found it useful to maintain a chart showing grading performance, a portion of which and covering operations in the "Hurd" cut is shown in Fig. 4. For this chart, sheets of ordinary cross-section paper were used. To indicate grading-machine operation, the sheets were ruled vertically into days of 12 hr., each horizontal square representing one hour. Normal working days of 10 hr. were from 6:30 a.m. till 6 p.m. with 1½ hr. for lunch at 11:30. The number of wagon loads was recorded in the field with a punch counting machine by the engineman on the tractor. This daily record, with due consideration of voids in the material as loaded, served for a rough check on the quantity of earth moved as figured from the cross-section sheets.

This work was under the direction of W. H. Coverdale, Consulting Engineer, 66 Broadway, New York, N. Y., who acted as Chief Engineer, and for whom the writer was Superintendent of Construction.

An Excavating Traction Grader

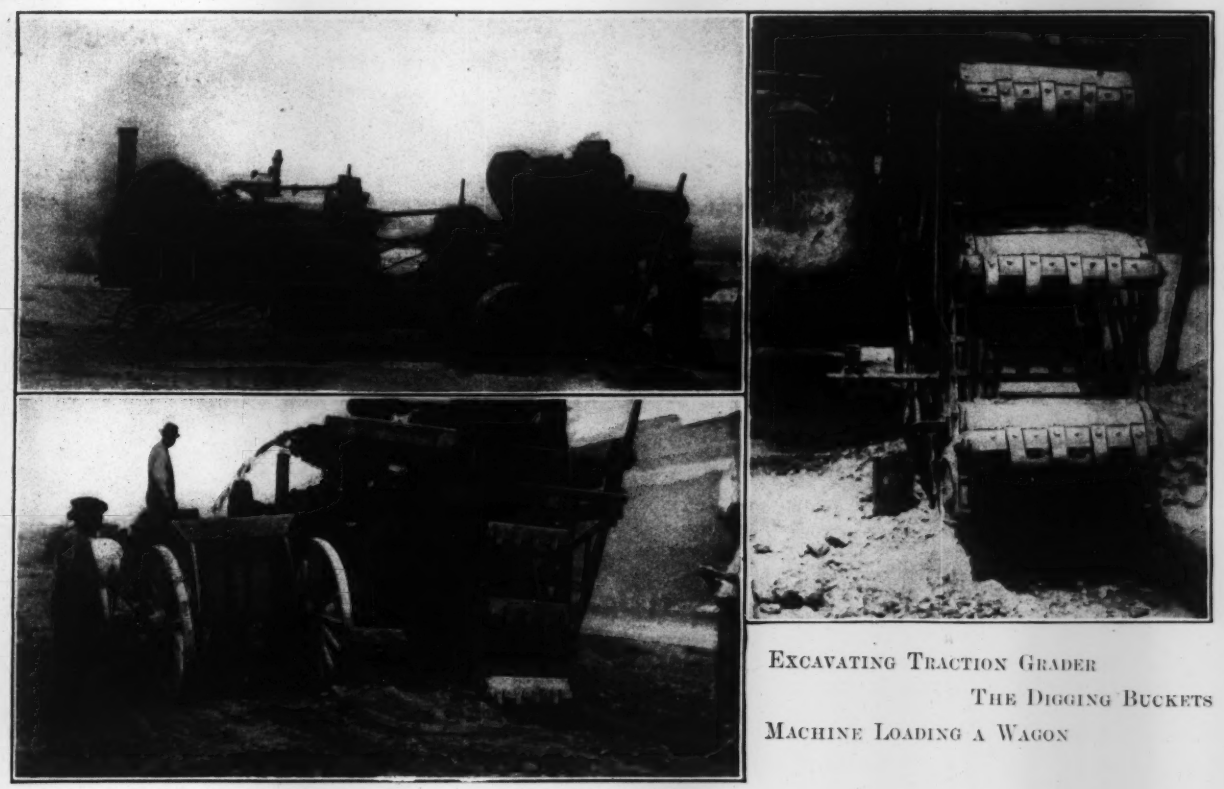
A machine specially designed and built for street-grading work, practically a heavy shallow-trench traction digger, is now in operation in Chicago. In this equipment, the excavating unit, a vertical flight of heavy digging buckets, carried on a triangular frame, is mounted on the rear of a long 4-wheeled truck. At the top, the flight of digging buckets has two points of support, carrying it almost horizontally over a transverse belt of steel slats, which distributes the excavated material to either side as required. On the truck is mounted a horizontal boiler with superimposed horizontal engine. The rated capacity of the machine is from 40 to 60 cu.yd. per hour.

Three principal elements make up the equipment, the digging and elevating unit, the distributing unit and the truck, power equipment, traction, steering gear, etc. The digging system embraces a flight of 11 heavy buckets 33 in. wide, with an individual capacity of ½ cu.ft. This flight revolves so as to dig upward and into the earth, and along the lower side works in an elevator channel of heavy steel plate. The digging buckets travel at a speed of about 140 ft. per min. The buckets are fitted with detachable and renewable teeth and cutting edges.

The buckets dig a trench 33 in. wide, 4 in. to 36 in. deep. The train of buckets is mounted off center, so that the outer edge of the trench is 8 in. outside of the tread of the rear traction wheel. This allows digging close up against a curb. The depth of digging is controlled by means of a rack and pinion on each side.

When digging, the grader moves forward automatically, an eccentric rod from the main countershaft and a dog and ratchet carrying a slow motion down to the rear wheels by sprocket and chains. The machine is driven forward at a speed of from 5 ft. to 20 ft. per min., depending on the material.

The transverse distributing unit, a belt built of steel cross-slats on rollers 11 ft. apart, is mounted so as to be fed by the buckets as they dump and, moving at about 300 ft. per min., shoots the material into wagons on either side. The belt is 18 in. wide, and the slats are turned



EXCAVATING TRACTION GRADER

THE DIGGING BUCKETS

MACHINE LOADING A WAGON

up at both ends so as to form a retaining channel. The conveyor can be run so as to load wagons at either side, as the work may require.

The bed of the truck, built of heavy steel sections, is about 18 ft. long. The rear traction drive is 48 in. in diameter with a 10-in. tread. The pilot wheels are 30 in. in diameter with the same tread.

A horizontal 25-hp. boiler is carried in the bed of the truck; superimposed upon the shell of the boiler, and rigidly bolted to it, is a 15-hp. horizontal engine, driving by chain to a transverse countershaft, located midway of the machine. The machine is driven from the countershaft by sprocket and chain.

The illustrations show this grader at work in W. 56th St., Chicago, on work under contract by C. F. Conway. The equipment was designed by J. C. Clark, and built by the H. B. Sackett Screen & Chute Co., Chicago. Mechanical details were worked out by P. A. Peterson of the Sackett company.

Tie Dating and Tie-Plates for Treated Ties: L. & N. R.R.

For marking treated ties with the date of laying, in order to keep record of their life, the Louisville & Nashville R.R. uses nails having the two last figures of the year stamped in the head. This is very general practice, but difficulty in keeping the records occurs sometimes through the head of the nail being broken or damaged so that the date mark cannot be read. To provide against this, a system has been adopted of indicating the date by the position of the nail in the tie, as shown in Fig. 1.

This road uses zinc-treated ties on tangents and creosoted ties on curves, and protects the latter against mechanical wear by means of metal tie-plates. The form of the tie-plate is shown in Fig. 2. It is of the shoulder type, and has two ribs on the underside, the depth of these ribs being sufficient to secure the plate against displacement but not sufficient to injure the wood or penetrate the more heavily treated exterior portion of the tie. The circular holes are for screws attaching the plate to the tie without relying upon the hold of the rail spikes.

For information as to the above we are indebted to W. H. Courtenay, Chief Engineer, and J. B. Lindsey, Superintendent of Treating Plants, Louisville & Nashville R.R.

FIG. 1 TIE DATING

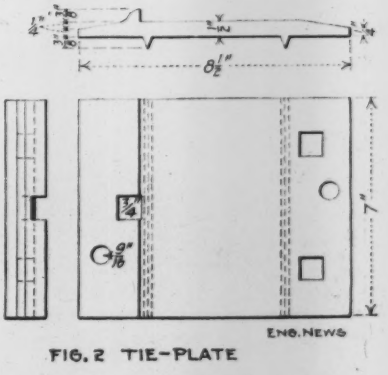


FIG. 2 TIE-PLATE

MARKING AND PROTECTING TREATED TIES ON THE LOUISVILLE & NASHVILLE R.R.

FIG. 1. SIDE VIEW OF HYDRAULIC FILL REPLACING A TRESTLE; SPOKANE, PORTLAND & SEATTLE RY.

A 40-ft. Railway Bank Made by Hydraulic Fill

Probably the highest railway embankment ever made by hydraulic fill is on the Spokane, Portland & Seattle Ry. along the Columbia River. It is made of fine sand pumped from the river. The elevation of track is 60 ft. above low water, and the fill has an average height of 40 ft. In Fig. 1, the partly completed bank is seen from below, while Fig. 2 is a view on top of one of the levels.

The dredge "North Bank" was designed and built for the embankment work. In Fig. 1, this dredge is just visible in the extreme left background. It has a wood hull, 130 ft. by 34 ft. There are two 250-hp. Scotch marine boilers, and a horizontal compound engine of about 600 hp. A full equipment of pumps, handling rigging, shop machinery, etc., is provided. The dredge has both jet and mechanical agitators, and a specially designed high-lift sand pump with 18-in. suction and discharge.

The slopes of the finished embankment are 1¾ to 1. The bulkheads for retaining the material are composed of 2x4-in. stakes, 4 ft. apart, on which are nailed 1x12-in. boards. The end bulkhead, built in this manner, is 30 ft. high.

Riprap was placed on the river side of the fill; the back side covered with cliff droppings to prevent the sand from blowing away.

The hydraulic method of making the fill secured three important advantages: cheapness, no interruption of traffic, and absence of settlement and maintenance when the fill is completed.

William Gerig, vice-president and chief engineer of the Pacific & Eastern Ry., supervised the construction of the embankment and designed the dredge.

FIG. 2. TOP OF PARTLY COMPLETED HYDRAULIC FILL

The Danger of Projecting Nails—The soundest and best thing to do is to educate the men who are responsible for leaving the nails in the board. Teach them the importance from a safety standpoint of pulling the nails out, or bending them over and hammering their points in, in a proper manner. It is safest, of course, to remove the nails entirely, but the next best thing is to hammer them in. But if they are bent over and hammered down, the work should be done in the right way. A nail bent so that its point remains a quarter inch or an eighth inch above the level of the board is still dangerous, because it is likely to tear the flesh or catch in the sole of the shoe, and if the end of the nail stands up by as much as a quarter or half inch, there is danger of the point entering the upper or soft leather of the shoe. In bending and hammering down a nail, the pointed end should first be turned over at right angles by means of pliers or the claw of a hammer. The body of the nail then is bent over and hammered down so that the pointed end enters the wood. All danger is then removed. [From "The Travelers' Standard," Travelers Insurance Co., Hartford, Conn., June, 1914.]

Raising a 240-ft. Cableway Tower to 280 ft.

The Tunkhannock Viaduct, now under construction on the Scranton-Hallstead line revision of the Delaware, Lackawanna & Western R.R., at Nicholson, Penn., is the largest and highest concrete bridge in the world. It consists of ten 180-ft. full centered concrete arches with two 100-ft. approach arches and has a total length of 2375 ft., with a height of rail above the lowest point of the valley of 242 ft. (see ENGINEERING NEWS, Aug. 28, 1913, p. 417). The construction of the viaduct is being carried out by means of two parallel twin-span cableways, with spans of 1525 ft. and 1510 ft., respectively, on either side of a central tower.

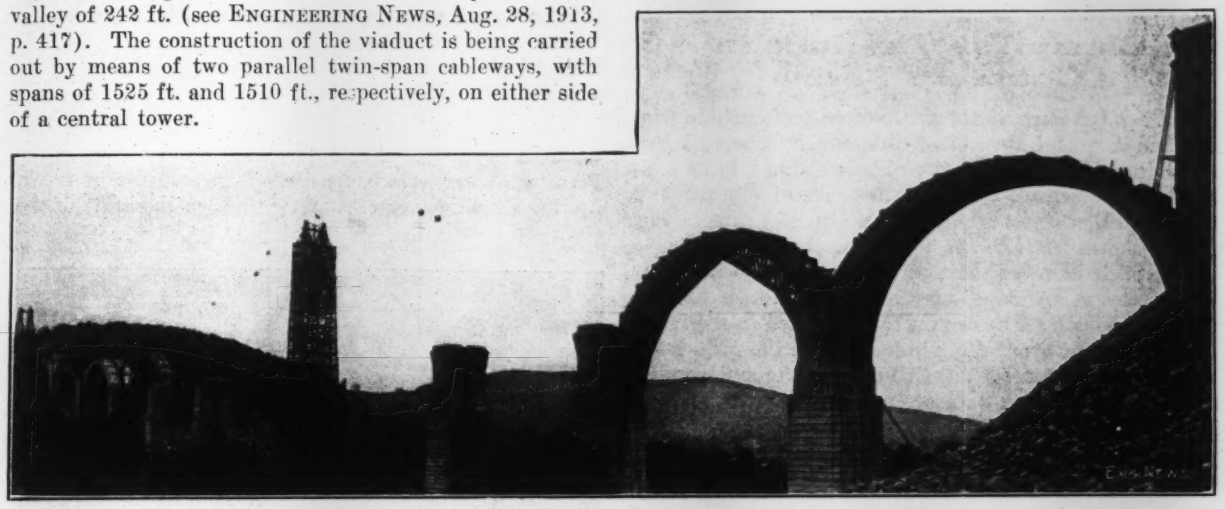


FIG. 1. THE TUNKHANNOCK VIADUCT, D. L. & W. R.R., UNDER CONSTRUCTION, SHOWING IN CENTER THE 280-FT. CABLEWAY TOWER

The main cables are each 2¼ in. in diameter and are fitted with high-speed carriages and shock absorbing fall rope carriers. Each cable was designed for an average load of 7 tons, but is being used to carry 10-ton loads in emergencies. The end towers are independent pyramidal structures cross-braced mainly at the top lift, with each part carrying at its top the sheaves for one cableway. The middle tower, however, is a thoroughly cross-braced single tower 40x60 ft. at the base and running up to a height of 240 ft. All towers are built of timber.

At the time the cableways were erected it was decided to build the center tower high enough only to give clearance for building the main piers and arches, but not high enough to complete the superstructure. However, after the work was started the contractor decided to use the cableway for the full construction of the bridge, which necessitated the elevating of the middle tower 40 ft. from its previous height of 240 ft. to a final height of 280 ft. The elevation of the tower was completed while work on the cableway was being carried on, with a short delay for shifting the cables to the new height.

In elevating the tower two separate pyramidal timber frameworks were built on top of the tower, while the cableway was in service, and on top of each of these frameworks was placed a new balancing device similar to one which remained in service on the 240-ft. elevation, and which is described below. When this elevation was completed, the cables were slacked off so that they rested on top of the viaduct, were then unshackled and their ends raised and joined through the sheaves of the new compensating device. According to the contractor, no difficulty was experienced in putting on the additional 40 ft. or in changing the cables. The tower was apparently as rigid as it was before the 40 ft. were added. The guys bracing the lower tower were left as before, and, in addition sixteen 1-in. guy lines were added to the new upper works. In the accompanying view (Fig. 1) the raised tower is shown with the two extra frames in place. The old pyramidal frames carrying the old compensating device were left in (in second lift below top of tower, Fig. 1).

The compensating device is shown in detail in Fig. 2. It consists of a structural steel frame securely bolted to the top of the timber framework and carrying, pendulum fashion from an upper pin, two steel plates through the bottom of which are pinned plates clamped to the opposite main cableways. The control and button ropes pass over sheaves at the top of the steel frame, around sheaves mounted near the bottom of the frame and back to the end towers. The pendulum pinplate is

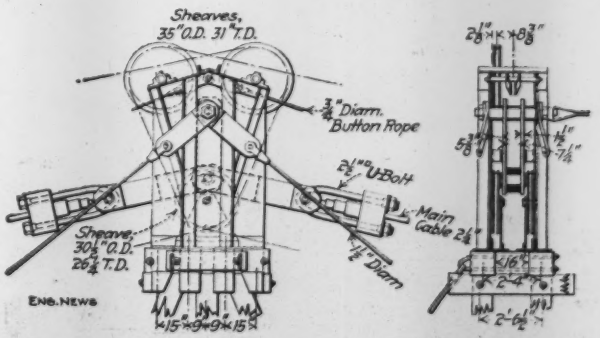


FIG. 2. COMPENSATING DEVICE FOR LONG SPAN TWIN CABLEWAY TOWERS. [PATENTED BY THE LIDGERWOOD MFG. CO.]

designed to permit an opposite cableway to take up the pull of an overloaded cableway on the other side and thus relieve the tower of the more severe overturning effort which would be induced by a rigid connection. Two of these balancing frames are mounted on the middle

tower, one for each parallel cableway, and they are guyed to each other and to the ground.

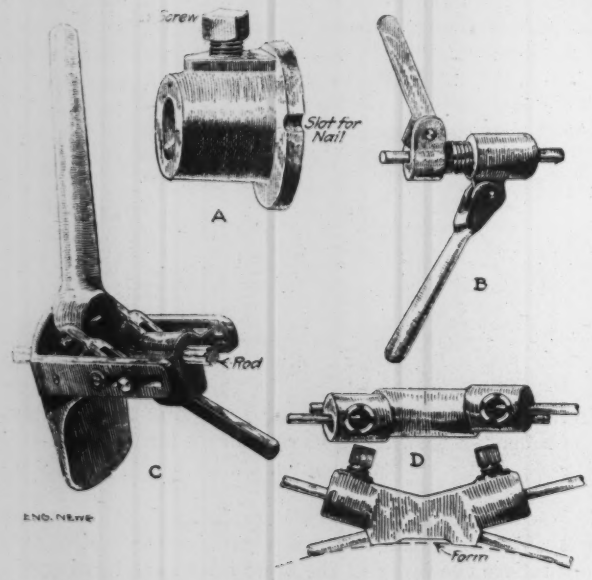
The bridge is being built for the Delaware, Lackawanna & Western Ry. under the direction of George J. Ray, Chief Engineer, by Flickwir & Bush, Inc. The cableway was designed and constructed by the Lidgerwood Manufacturing Co., and the patented compensating device was designed by W. J. A. Rankine of the company.

Clamp for Tie-Rods in Concrete Forms

Rod ties for concrete forms are often preferable to wire ties, and a special clamp fastening which is being introduced to hold and secure the rods is claimed to be simpler and more effective than a threaded rod with nut and washer, while it is applied in much less time. The clamp, shown at (*A*) in the accompanying cut, is a sleeve which fits over the rod, and is locked to it at any position by means of a set screw which forces the rod against a V-slot. The clamps can be nailed to the forms to prevent them from being lost. Should the form bulge or spread, a tightening wrench (*B*) is put over the end of the rod and operated as a screw jack to force the form into place.

In many cases the rods are inclosed in sleeves, but for cases where they are embedded in the concrete, a special pulling device (*C*) is used to remove the rods. This grips the projecting end of the rod and has a lever by which a powerful pull is exerted. If used before the concrete has taken a complete set, this will remove the rods without bending them and without defacing the surface of the concrete. If the rods are to be left in for any length of time, they should be jarred so as to loosen the grip of the concrete, and then they may be removed when convenient.

The clamp may be used also with outside tie rods, as in forms for columns and girders, and a special double clamp and saddle (*D*) is used to take both ends of hooped clamp rods on circular columns.



Clamp Fastening for Tie-Rods in Concrete Forms
(A, clamp. B, wrench for tightening rods to straighten forms. C, rod puller.)

These devices have been used extensively in building construction work, and also on the retaining walls of track elevation work in Chicago. They are made by the Universal Form Clamp Co., 1214 W. Madison St., Chicago.

Putting a Sewer On the Map by a Subterranean Survey

By J. E. Schwaab*

The City of Alton, Ill., is contemplating the installation of a sewer system to take care of that part of the city known as North Alton. The only feasible outlet for this sewer system is the present main sewer known as the Piasa St. sewer, which consists of an arched-over creek, bearing a similar name, running through the heart of the city.

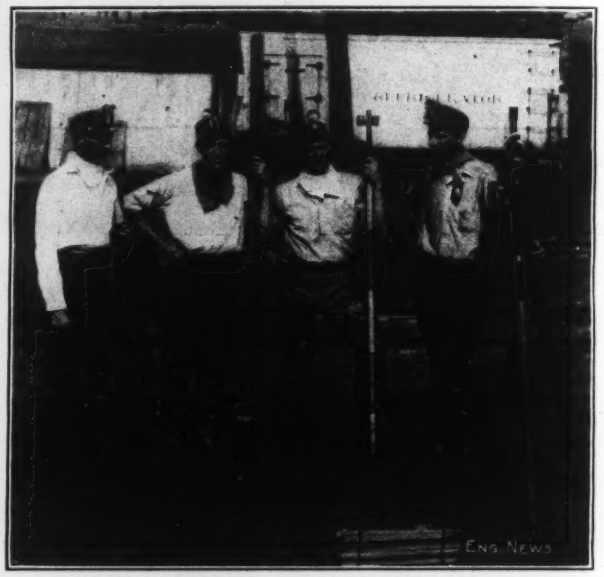


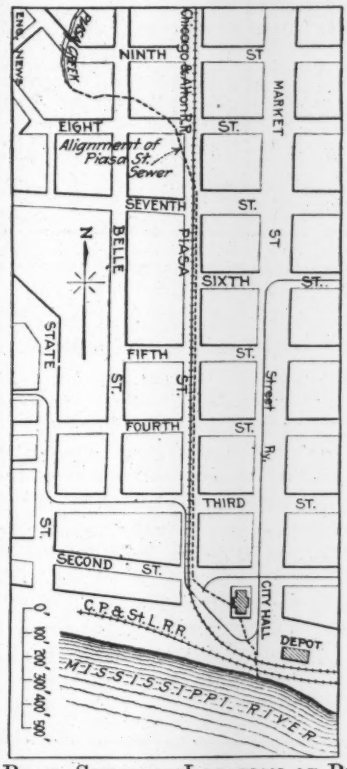
Fig. 1. Survey Party about to Start a Subterranean Sewer Survey, Alton, Ill.

This creek was arched over in 1858 but no record was made of its exact location with reference to present streets. Since then, several extensions have been made, making a total of 2978 ft., which have been arched over with arching varying from 9 to 12 ft. in span. It is now proposed to extend the arching over of this creek to North Alton for the main outlet for the proposed North Alton sewer.

This fact has given Assistant Engineer W. C. Vincent the rather unique experience of making a survey of the present Piasa St. sewer, to determine if the size, location and condition of same would warrant its being used as the outlet of the proposed sewer.

The method of procedure did not differ essentially from the ordinary mine survey, but had the additional feature of working in running water and sewage. At the outlet into the Mississippi River, the water had backed up into the sewer for a distance of 300 ft. This necessitated

*City Engineer, Alton, Ill.



SKETCH PLAN, SHOWING LOCATION OF PIASA ST. SEWER, ALTON, ILL.

the use of small boats. Hip boots proved ample in making the balance of the survey.

The task was accomplished in the following manner: (1) Transit points were established in the center of the sewer at each angle point. This was done by building up a pile of stones high enough so that the top stone protruded above the surface of the water. (2) Starting at the outlet of the sewer as 0.0, the distance along the center line was chained and pluses taken to the transit points as above described. (3) Angles and levels were obtained. This was one operation, levels being taken with the transit while securing the angles. This avoided an extra trip through the sewer.

The first set-up was made at the outlet, a sight taken on the transit point at the first angle and a backsight taken on the opposite side of the Mississippi River. The next set-up was made at the first angle point, a backsight taken at the outlet and a foresight at the next angle point. This was continued throughout the entire length of the sewer. Miner's lamps were used throughout the work, by which the rod sights were easily obtained.

The alignment and levels thus secured were rerun on the surface, as a location of the sewer was desired in respect to the city streets; and it was particularly desirous to know if it ran under the City Hall Building.

Quite a few surprises resulted as to the actual location of the old sewer. It was found that the sewer ran under the southwest corner of the City Hall at a point directly underneath a very bad settlement of the south wall of the building. This wall is now in a precarious condition.

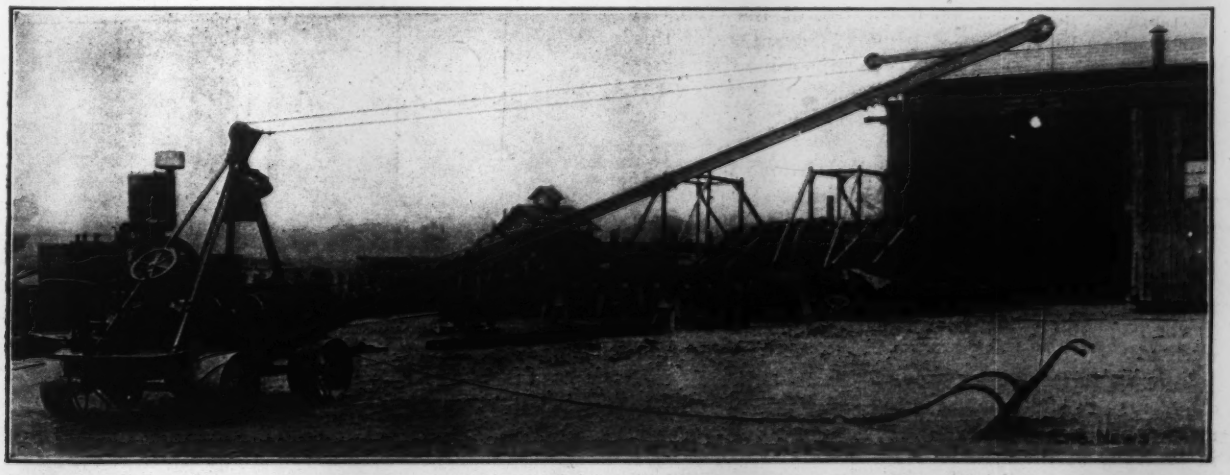
A feature not usually encountered on surveys was the vast number of bats. In several places they could be seen swarmed up on the roof of the sewer like bees. When they were disturbed by our lights they caused a commotion which was not particularly agreeable.

On a whole, the task was not as disagreeable as one might anticipate, yet it must be admitted that the fresh air and sunlight were greatly appreciated at the end of the day's work.

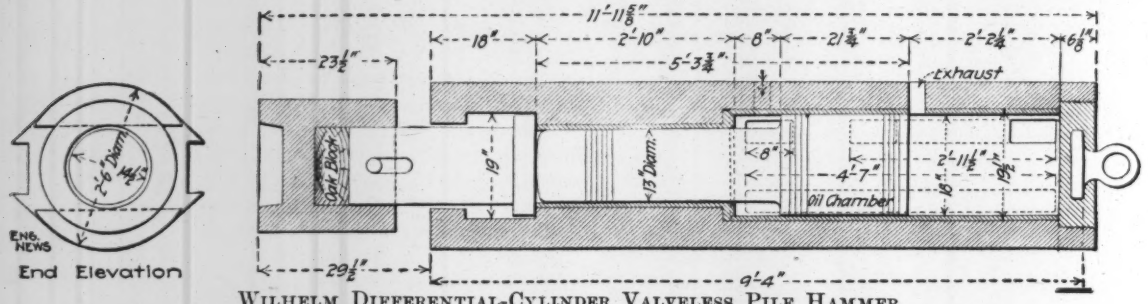
A Drag-Line Backfilling Machine

In pipe-trenching work where machines are used for excavation, the cost of backfilling (by hand) is frequently higher than that of excavation, and there is opportunity for considerable saving in this item by the use of machines instead of hand labor. The backfilling machine shown in the accompanying cut operates on the drag-line principle, the tractor traveling along the side of the ditch opposite to that on which is the spoil bank, and hauling the scraper to and fro.

The tractor has a frame 7x10½ ft., mounted on four 30-in. wheels with 18-in. tires, giving a wheelbase of 5½ ft. and a width of 5 ft. 9 in. over wheels. At one side is an A-frame, from the head of which is guyed a boom, giving a maximum reach of 22 ft. A 10-hp. gasoline engine drives a double-drum hoisting engine, and



A DRAGLINE BACKFILLING MACHINE



WILHELM DIFFERENTIAL-CYLINDER VALVELESS PILE HAMMER

also the propelling gear. The clutches of the two drums are operated by pedals, leaving the operator's hands free to work the steering wheel and the lever of the propelling-gear clutch. The boom is in four sections, put together with bolts, so that its length can be varied as required. The heel section is of V-shape, with broad base, and the head section carries the guy attachment and the sheave of the hoisting cable. The weight of the machine is 5500 to 6000 lb. Its traveling speed is 2 m.p.h.

The scraper is handled by a loading or drag line and a hoisting line. When in position behind the spoil bank (as shown), the loading line is hauled in, dragging the scraper to the edge of the trench, while the hoisting line is paid out. The latter line is then hauled in, pulling the scraper back, while the loading line is paid out. The scraper is set in position by hand, as shown, and the man may or may not keep hold of it, according to the requirements. A broad scraper is shown, but any kind of scraper bucket may be used.

The machine has worked at the rate of five or six strokes per minute in handling heavy clay that had been exposed for several months and had became practically a solid mass. it is designed for work with trenches up to 6 ft. wide and 20 ft. deep. It is built by the F. C. Austin Drainage Excavator Co., Railway Exchange Building, Chicago.

A Valveless Automatic Pile Hammer

A new pile hammer is just being put out which has the remarkable feature of containing no valve. The piston itself acts as its own valve, by allowing the working fluid (steam or air) to flow through bypass ports from the ring-shaped space below the head of the piston into the main cylinder area when the top of the stroke is nearly reached. The great point of advantage of this hammer is its extreme simplicity; at the same time practical tests up to the present have indicated that it works smoothly and effectively, so that it is likely to prove an ideal contractor's tool.

Besides its extreme simplicity, an obvious feature of the hammer is the absence of any thin rod connecting the piston and the ram or striking point. This means the elimination of liability to breakage, and positively brings the full weight of piston and ram into action. On the other hand, the steam efficiency of this hammer is below the best steam-engine requirements. The hammer will work with air or steam.

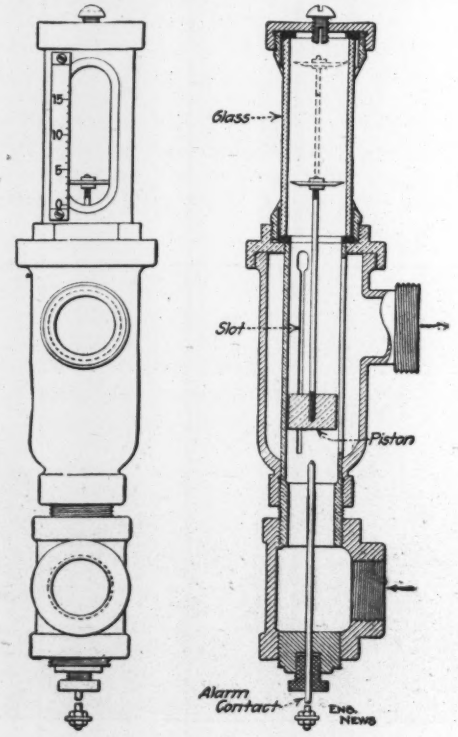
It needs to be noted in addition to what the drawing shows that two oil chambers run the full length of the main cylinder at two opposite points. These are formed in the body of the cylinder wall and are covered by the lining, but supply oil to the piston by means of a number of small oil holes through the lining. These oil chambers can be filled from the outside of the cylinder; they hold enough oil to last three or four days.

The pile cap forms an integral part of the hammer in this construction. An oak cushioning block is placed between the body of the cap and the plug or die. The latter projects into the end of the cylinder body and is held in a cross-slide in position to receive the blow of the hammer. The arrangement is fully explained by the drawing.

This new hammer was invented by J. E. Wilhelm, Cleveland, Ohio. It is being built by the Contractors Machinery Co., 2511 Elm St. N. W., Cleveland. The sizes which have been built up to the present include a 275-lb., a 500-lb. and a 1000-lb. hammer. A 5-ton hammer is now being built. The smaller hammers strike about 200 blows per minute, the large hammers probably 150 or more.

NOTES

A Simple Flow Meter for indicating the flow of liquids in pipes has been designed and patented by J. F. Vaughan, of the Stone & Webster Engineering Corporation, Boston, Mass., which is applicable to a large number of piping systems, such



A SIMPLE FLOW METER

as cooling, heating, lubricating, refrigerating, boiler supply, etc. The instrument is very simple in theory and construction; it consists merely of a cylindrical chamber, with suitable pipe connections, enclosing a slotted tube with a sliding piston, on the top of which is a rod and scale indicator. When the liquid enters the lower orifice (see accompanying sketch) it enters the slotted tube through the bottom and pushes up the piston in proportion to the amount of flow, or until the orifice is large enough to allow all the flow to escape. Above the flow-measuring tube is a glass tube enclosing the piston rod with its indicator and a scale graduated to gallons per minute. An alarm system for calling attention to a stoppage or a fixed minimum of flow is provided by a brass rod set through a stuffing box at the bottom of the cylindrical chamber; on the descending piston coming in contact with the rod an electrical circuit with a lamp or bell is made. The contact rod may be adjusted to project any length into the chamber. The flow measured is proportional to the area of the slots in the cylindrical chamber, which in turn is proportional to the rise and fall of the piston. It is claimed by the makers that under test the indicator gives practically a straight line calibration over a large range, and the loss of head in operating the instrument has been found negligible, not exceeding 0.04 lb. per sq. in. with a capacity of 15 gal. per min. Screens may be used in connection with the meter, where the water or other liquid is dirty. The meter is easily cleaned by unscrewing the top and slipping out the slotted tube and glass. The Vaughan flow meter is made by W. & L. E. Gurley, Troy, N. Y., and is listed at $20 for 20-gal. capacity and $50 for 100-gal. capacity.

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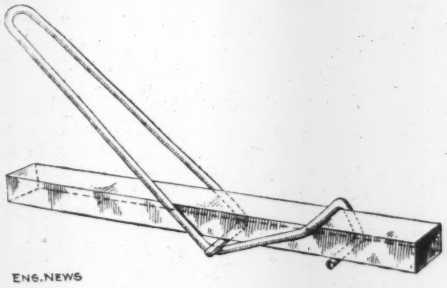
A New Stirrup for use in concrete reinforcement can be snapped onto the main bar very quickly and easily, and holds itself in place by a spring-friction grip. These results are secured by the peculiar bent shape of the lower end of the stirrup. The sketch explains it.



These stirrups are made (stock sizes) of ¼- and ⅜-in. round rod, in various lengths, and are intended to go on bars of ¾- to 1¼-in. size. The Diamond Expansion Bolt Co. (90 West St., New York), is the maker.

To place the stirrup on the bar, the forward hook end is slipped around the bar, and then the rear loop spread apart, dropped over the bar, and locked together again.

⁂

Breaking Up a Sunken Steel Barge—The drill barge "Teredo," a steel hull 112 ft. long, 36 ft. beam and 8 ft. deep, was sunk in the Culebra Cut on the Panama Canal by the disastrous explosion of July 20 (noted in Engineering News, July 23, 1914, p. 223). The barge was so damaged as to make impracticable its raising intact so it was broken to pieces under water and raised in sections. This breaking up was done by blowing apart the steel plates with dynamite placed in old fire hose, laid along the lines which it was intended to cut. The "Canal Record," July 29, 1914, states that by this method a clean cut was made in the plates and the sections easily lifted out.

⁂

The Problem of Durable Bridge Flooring for Country Bridges—In very many cases the condition of bridges on state and county highways is not at all in keeping with the character of the highway. In the case of new construction, so far as possible, new bridges should be provided, which will be of a character corresponding to the highway improved. In cases where this is impossible existing plank floors should be replaced with some form of improved flooring, either concrete, if the structure is of sufficient strength to carry it, or creosoted blocks. This will in the end be found to be an economical measure as the increase in travel induced by the improvement to a highway subjects old bridges to an additional strain which many of them are not calculated to withstand. If such bridges were removed to some location where traffic is not so trying, and a modern bridge built at the point of improvement, both structures would last for many years.

The matter of providing satisfactory floors for those bridges which are not of sufficient strength to carry a concrete floor in addition to the live loads which they may be called on to carry, is a very serious problem. The annual cost of renewing ordinary plank floors is very great and is constantly increasing. Also it is yearly growing more and more difficult to get plank which will make a satisfactory floor. The loose, warped and twisted floors, which are getting to be so common, are a source of great annoyance to travelers and a big expense to the towns which have to maintain them. Treated wood blocks are recommended, for these may be used on many bridges whose strength will not permit the use of concrete, and when once properly laid are very nearly indestructible.

On those bridges where the cost of wood blocks is too great, a very satisfactory floor may be made by 2x4-in. scantling set on edge, spiked together one by one as they are laid, and finished with a surface coat of hot tar and sand, fine gravel or stone screenings. This makes a smooth, noiseless and durable floor and is very much cheaper than plank laid in the ordinary way. ["New York State Highway News," July, 1914.]

⁂

Orienting Oneself in the Heart of Boston—The accompanying picture is not a wall. It is a view of the sidewalk at the feet of the photographer. The sidewalk is at the corner of Hawley and Franklin St., in the city of Boston, Mass. The streets of Boston are said to follow the cowpaths of colonial days. Every uninitiated traveler in Boston believes this is true.

We don't know who is responsible for the brass marker shown in the sidewalk at the bottom of the accompanying illustration, but it is a pity the city government of Boston does not adopt the scheme generally, for even Bostonians are hazy on the points of the compass. The new Custom House tower, forming a conspicuous landmark, is said by natives to be in a different place on the horizon, when viewed from certain locations, than that which they would have naturally supposed.

BRASS MERIDAN IN SIDEWALK, FRANKLIN AND HAWLEY ST., BOSTON, MASS.

The proper marking of street corners is a part of the municipal public work which is frequently overlooked. One fault to be found with Boston is that many street corners are not marked with the names of the streets, or at least there is no uniform method of posting signs. The design and erection of street sign posts, or sign and lamp posts combined, is a legitimate part of a city engineer's work, and is just as important to a well ordered city as clean streets.

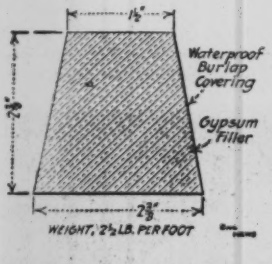
⁂

Pockets for Small Surveying Instruments — A correspondent, Frederick W. Salmon, of Birmingham, Ala., has found a convenient method of transporting various small instruments on surveying and other trips. These instruments include a watch with hand reading to ½ sec., for estimating distances by sound, flow of water, etc.; a passometer or pedometer, preferably the passometer; a pocket azimuth compass or pocket sextant with clinometer, for most pur-

poses the azimuth compass with a reflector sight preferred; a pocket aneroid barometer compensated for temperature; a chartometer reading to 25 ft. and to $\frac{1}{8}$ in. on the measuring wheel; a 25-ft. steel tape in a leather box. To comfortably transport these six instruments Mr. Salmon has his pants made with six "watch pockets," below the belt line, three on each side of the center line. These pockets have flaps over them and buttons, to ensure that nothing will be lost from them in boating or riding horseback. By having the pockets made when the clothes are made there is no extra expense. The pockets are a little larger than the ordinary "watch pockets," about 3½ to 4 in. wide at the mouth. Mr. Salmon finds that the instruments "ride better in this location than in vest and coat pockets."

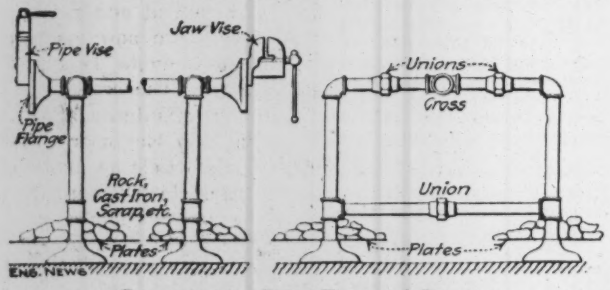
⌘

A Composition Gypsum Nailing Strip or Floor Sleeper has recently been developed by the U. S. Gypsum Co., Chicago, Ill., to be used in place of the usual wooden nailing strip in concrete floors or walls. Timber embedded in concrete is very apt to decay in a short time or at least to swell when damp, and thus crack the concrete. The composition material should be free from both of these defects. The "nailing screeds," as they are called, are made in standard 6- and 8-ft. lengths of the section shown herewith. The material is a compound of gypsum and sawdust sheathed with a heavy burlap fabric. Each section is molded under pressure, and, after the filler has set, is kiln-dried, waterproofed, and cut to length. The material takes and holds nails.



⌘

A Convenient Portable Pipe-Fitting Bench which can be quickly knocked down or set up by unscrewing or screwing six unions is illustrated by the accompanying cut, sketched



COLLAPSIBLE PIPE-FITTERS' BENCH

by a reader from a bench in actual use, but improved by him in a few details. The legs are loaded with heavy rock or this device for stream-gaging, while in the Albany office of the U. S. Geological Survey. He claims it to be especially useful in gaging streams which have wide daily fluctuations. scrap material as shown to make the bench firm. The dismantled bench occupies but little space. Boards can be placed lengthwise on the top of the bench, making it adaptable to many uses.

⌘

Efficiency Chart of Car-Loading Gang—With the accompanying chart, the efficiency of a car-loading or unloading gang can readily be estimated by the foreman, when the cost of the day's operation and the number of cubic yards handled are known. Efficiency in the following three operations can be computed: (1) unloading cinders; (2) loading rock ballast; (3) right-of-way ditching. As an example, suppose a certain

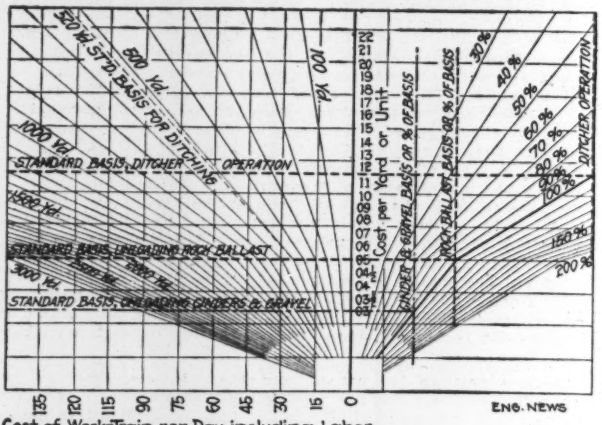

600 Yd.
1000 Yd.
1500 Yd.
3000 Yd.
STANDARD BASIS, DITCHER OPERATION
STANDARD BASIS, UNLOADING ROCK BALLAST
STANDARD BASIS, UNLOADING CINDERS & GRAVEL
Cost per Yard or Unit
ROCK BALLAST, BASIS OR % OF BASIS
CINDER OR GRAVEL, BASIS OR % OF BASIS
DITCHER OPERATION
Cost of Work-Train per Day including Labor and Foremen, Train and Eng. Crews and Rent of Machinery and Cars
ENG. NEWS

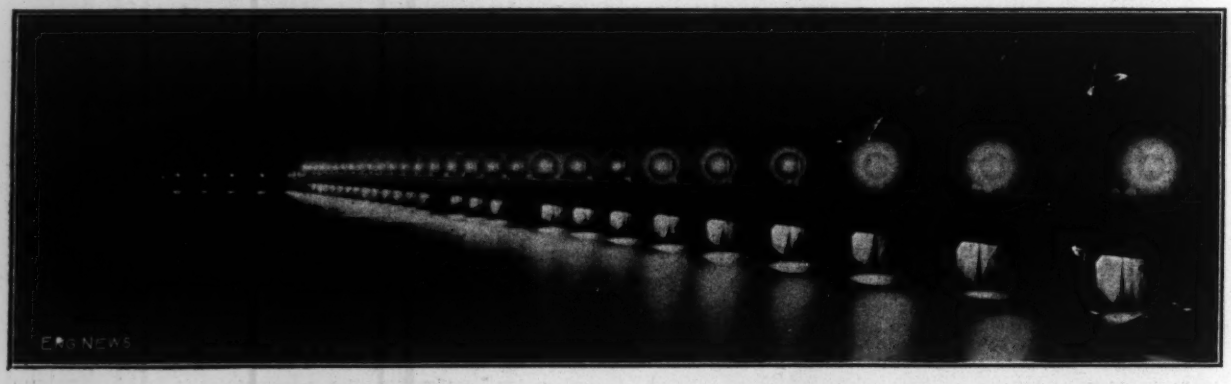
CHART COMPARING STANDARD BASIS WITH WORK-TRAIN OPERATION

day's expense was $60 and the quantity of ditching handled 400 cu.yd. From the figure 60 on the abscissas, rise vertically to the intersection with the diagonal 400-cu.yd. line; move horizontally to the center line of unit cost; finally (still horizontally to the right) to the vertical line representing the operation performed, at which point the efficiency is indicated by the intersecting diagonal. In the case considered, the unit cost was 15c. and the efficiency 76%. The standard basis of ditching right-of-way is 11½c. per cu.yd.

This chart was prepared by H. A. Lloyd, Erie R.R., Jersey City, N. J., and published in the "Bulletin" of the American Hoist & Derrick Co.

⌘

Lighting a Dam at Night—The accompanying figure shows the artificial illumination provided at the new Coon Rapids Dam of the Northern Mississippi River Power Co., some six miles above Minneapolis. The structure is 2070 ft. long and has a 1000-ft. spillway surmounted by a footbridge on piers and 28 Tainter gates, 33x8 ft. A line of 100-watt tungsten lamps has been placed on goose-neck standards, one at each gate stand, to expedite foot passage and to facilitate, in dark hours, the work of handling the gates and clearing away ice and logs in some seasons. The Coon Rapids development was described in "Engineering News," July 16, 1914, and a daylight view was shown of the dam.



NIGHT LIGHTING OF DAM ON MISSISSIPPI RIVER AT COON RAPIDS

Editorials

The editorial and publication offices of ENGINEERING NEWS were moved on Sept. 1 to the recently completed Hill Building, at the northwest corner of 36th St. and Tenth Ave., in New York City. The building, which has been especially designed in its structure and equipment for the use of the Hill Publishing Company, is easily accessible from any part of the city by transfer to the westbound 34th St. cross-town surface cars, which pass the building. The editors of ENGINEERING NEWS will be glad to see any of their friends at their new home.

Five Successive Bridges on One Site

The article in this issue on the Farmville Bridge of the Norfolk & Western Ry. is of especial interest to the bridge engineer because the five successive structures which have been erected on this site since 1853 typify to a fair degree probably the changes in bridge engineering practice which have taken place within the memory of engineers still living and in active practice. The original wooden structure, at the time claimed to be, combining length and height together, the greatest bridge in the world, cost only $167,500, or about $50 per lin.ft., of which probably two-thirds was the cost of the brick piers.

It is noteworthy that this wooden structure was in service for 17 years, a longer life than any of the structures which have succeeded it.

The Fink trusses of 1870 were replaced in 1886; but curiously enough the structure of 1886 weighed less per lineal foot than the structure it replaced, though it was supposed to carry nearly twice as heavy a rolling load. Those were the days when bridge builders and designers were exercising ingenuity in saving metal. It is noteworthy that the 1886 structure required strengthening at the end of fifteen years, as did a good share of the bridges built about that time.

Direct comparison of the weight of the early bridges with the present structure cannot be made since the early bridges were trusses resting on brick piers, while the structure just completed is a steel viaduct. It is at least interesting to note, however, that while the 1886 bridge weighed only 540 lb. per lin.ft., the new steel viaduct weighs 2900 lb.

A Record Penetration for Concrete Piles

A few months ago, we called attention in these cloumns to the extreme length to which it is possible to cast reinforced-concrete piles, citing then the new docks at Havana and at Halifax, at each of which piles nearly 80 ft. in length were uniformly used. In both of these structures, however, as in most places where such extreme pile lengths are used, the piles were driven in deep water, so that nearly half the pile is above ground. While this condition increases the strength requirements of the driven pile, because of the necessity of designing for the unrestrained column, at the same time the shattering action of driving the entire 80-ft. length is avoided. In the abutment piles of the Farmville High Bridge, which are described on page 522 of this issue, the condition is reversed, and the entire 60-ft. reinforced-concrete pile is driven through an old fill, which makes extremely hard driving.

So far as our records and recollection go, these piles are the longest reinforced-concrete piles ever driven for their full length through ground. Testimony of the engineer in charge that the average penetration throughout the whole length was only ¾ in. per blow, and that the final penetration was from 1-30 to 1-40 of an inch, is sufficient evidence of the solidity of the old fill. That the piles stood up under such battering does great credit to their design and construction and incidentally is a testimonial as to the strength of the reinforced-concrete precast pile in general.

Lessons from Some Field Tests of Concrete

It must be that a number of government and private corporations engaged in concrete work are now making consistent and regular field tests on concrete, but very few of such tests have been reported. The scarcity of figures of this sort makes all the more interesting the table given by E. F. Wallace of the City Engineer's Department at Kansas City, Mo., in his article on another page in this issue on field tests of concrete for municipal work at Kansas City. At the same time the figures provoke certain thoughts regarding the application of such tests to the concrete designer's art.

We believe that the argument in favor of field tests is that it brings to the testing machine the precise material about which information is desired. A laboratory-made concrete (or even field-made concrete for laboratory purposes) must necessarily be of a higher grade than that made by the workman without any idea on his part that it is to be used for a test. In spite of this obvious fact, the lessons learned from any field test have more of an indirect effect on concrete making generally than a direct effect on the structure under test because, under present conditions, by the time the results of the tests are known, it is too late to change or remove any concrete that may fall below specification.

Suppose, for instance, at the end of 28 days, the blocks taken from a certain structure may test in compression only 650 lb. per sq.in., as with a number of the blocks in Mr. Wallace's table. By that time it is quite possible, in fact it is very probable under the usual methods of construction, that the particular concrete from which the sample was taken would be sustaining a load fully equal to the load for which it was designed under an ordinary specification unit stress of, say, 650 lb. per sq.in. What can the designer or the constructor do? By this time this concrete is buried far down in the structure or is at least so far back in the structural process as to make its

removal extremely difficult if not impossible. The sole advantage gained from the test is an indication to the engineers that either such concrete should be designed for unit stresses much lower than those used or that, in future, measures should be taken to use different methods or materials in making concrete. A byproduct of the test will be the serious concern that the designing engineer ever after has for the safety of this particular structure.

An especial instance of the fallacy of this kind of testing comes to mind in the recollection of the design of a monster concrete bridge which some years ago was widely advertised as projected in New York City. This bridge was to be designed under the assumption that the concrete in it would stand at the end of thirty days a unit stress of 3000 lb. per sq.in. in compression. At the time of this announcement, we were curious to know what the engineers would do if at the end of 30 days they found that certain concrete in the middle rib of this bridge, for instance, had stood only 2000 lb. per sq.in. Would they have torn out the whole arch-rib until they reached this particular concrete, or would they have posted notices on the finished structure stating that only two-thirds of the designed load was allowed to cross the bridge? Obviously, to make field tests of maximum utility, some very accelerated test must be devised.

A second and possibly more important feature of Mr. Wallace's test is the extremely low value that the field concrete attained. It is quite common to assume that 1:2:4 concrete at the end of 30 days is good for 2000 lb. per sq.in. in compression. On the basis of this assumption the unit compressive stress in concrete has been consistently boosted through the past ten years from 300 lb. per sq.in. to 650 and in some cases to 800 lb. per sq.in., and it is within our knowledge that certain enthusiastic supporters even claim 1000 lb. per sq.in. is not excessive. In contradiction to this, our readers will remember that W. A. Aiken, an experienced concrete tester, stated, at the recent meeting of the American Society for Testing Materials, that the crushing strength of broken stone or gravel concrete made up under ordinary field conditions will not average over 1500 lb. per sq.in. at the age of 30 days.

The Kansas City test, however, on actually made concrete shows that even this figure is high, for the 1:2:4 concretes in the table run from 403 lb. to 1657 lb. with an average of 954 lb. per sq.in. at the end of 28 days and even at the end of a half year only two out of the lot reached a higher value than 2000 lb. per sq.in. With the somewhat leaner mixtures, which were quite probably as rich as much nominal 1:2:4 concrete, the unit stresses are very much lower.

Should not such figures serve to indicate to concrete designers that the time has about come when unit stress in concrete cannot be boosted much higher? The Kansas City concrete was practically all made for structures where great strength is not particularly necessary and undoubtedly on this account the construction and the inspection dropped below what it would have been in a high-class structure, but the remarkably low figures obtained and the perfectly obvious lack of uniformity in the stresses obtained from the different takings from the same batch should at least give pause to the bolder of designers and building-code makers. Field concrete is not laboratory concrete, and ordinary field concrete is not even comparable to carefully made field concrete. The adoption of a high unit stress predicates the utmost care in the supervision of the concrete making and placing.

The Business Situation and the Outlook

For a month past two subjects have absorbed the public interest almost to the exclusion of all others. The first is, of course, the gigantic struggle now in progress in Europe. The second is the effects of that conflict upon the United States. Great as is the interest felt here in the daily reports from the vast battlefields of Europe, the people of this country are most concerned with the effects of the war which they themselves experience. And the most important of these effects is not the great increase in the cost of living, due to the rise in price of all food products, heavy though the burden that brings upon all classes, from richest to poorest. More serious than this is the widespread paralysis of business, taking away employment and the means of subsistence from hundreds of thousands, and this at the very time when their small savings are most rapidly used up by the high cost of the necessaries of life.

There is probably no industry which is more severely affected by the present financial paralysis than the business carried on by engineers and contractors. Construction work on new enterprises either for corporations or cities is almost invariably carried on by borrowed money, hence cutting off the supply of funds puts an immediate stop to new work.

It is a question of greatest interest, therefore, how long before the financial machinery of the world will be again set in motion. By what means and from what source can money again be obtained to finance the important construction work now suspended all over the country?

There can be no doubt, as we pointed out in these columns a month ago, that higher rates for investment capital is one of the certainties of the future. While the war continues, those European countries which have been the chief reservoir for investment capital to supply the rest of the world, will require all the capital their people can furnish to take care of their war expenditures. When the war ends, Germany and France and England will need vast amounts of capital to make good the destruction wrought by the war and to rehabilitate their industries.

The investors of the United States, therefore, are the chief source to which borrowers must resort for a long time to come, not alone to carry on work in the United States itself, but in South America and Africa and Asia and Australia as well.

For the moment, it is true, investment business in the United States is almost at a standstill, due first to the stoppage of a large part of our important financial machinery, such as the stock exchanges; but also to the general lack of public confidence. And the last cause is as important as the first. Until the public realize that this vast war catastrophe, after all, affects only a small part of the world's inhabitants, and that the great mass of its people are going on as before, producing and selling and buying, furnishing a vast market for food, and clothing and manufactured goods—in short, until the public realizes that this great world calamity, with all its evils

and losses will also furnish great commercial opportunities to the United States, public confidence here will not be fully reëstablished.

On the other hand, when this happens—when the dam that now holds back industry is broken, a flood of phenomenal business activity may follow. It was so during our own Civil War. Vast though the destruction and disaster that then went on within our own borders, the paralysis of business at the outbreak of the war was followed by a business boom that made great profits for factories throughout the North.

The foundation conditions now exist that may bring about such a wave of business activity. The nations of Europe are eager to obtain the food products of North and South America, which are as necessary to their military operations as arms and ammunition; the price is a secondary consideration. This means hundreds of millions of dollars additional profits to the farmers of the United States and to the ranchers of Argentine. Merchants in Asia and Africa and South America are already seeking to purchase in the United States, the cloth and hardware and other merchandise which they are unable to obtain from Europe. Here again prices will be fixed on a more liberal scale than they have been under the stress of international competition. They will doubtless have to because the higher cost of living will sooner or later make necessary an increase in the wage scale.

And at no time in our history have the possibilities of raising in the United States a vast amount of capital for investment been anything like so favorable as today. A quarter century ago, an enormous amount of Eastern capital was pouring into the states from the Mississippi River to the Pacific to transform a region still virgin into farms and towns and cities. That region is now itself wealthy and is not only furnishing to a large extent the capital required for its own further development, but is buying bonds and stocks issued elsewhere.

As our own construction news columns show, engineering work in the United States is not by any means at a standstill, notwithstanding the European war. Cities are going ahead with necessary extensions of sewerage and water-supply and street-paving systems. States are proceeding with highway improvement. In general, the financing of these works just now is being taken care of directly in the locality affected. Trust companies or savings banks or the city's sinking-fund commission take the bonds, where bonds are issued, or the work is carried on by the proceeds of a tax levy or from assessments on the property benefited.

There is no doubt that the financing of such work could proceed on a much larger scale were only the financial exchanges open so that a price for investment securities might be established. At present investors are loth to purchase stocks or bonds lest they use up their available capital and find themselves unable to purchase if some more favorable opportunity is offered when business on the Exchanges is resumed.

The volume of investments could also greatly increased, it is probable, were better means adopted to reach the small investor. During the Civil War, the firm of Jay Cooke & Co. accomplished wonderful results in raising capital for the Government's necessities by popularizing the sale of Government bonds. In the Spanish-American War of 1898, the Government's offer of a 3% loan directly to the public met with an amazing response.

It must not be overlooked, either, that the ruling rate for capital has an important influence upon the amount of capital which can be obtained. Let safe bonds at six per cent. or seven per cent. be open to public subscription, and large amounts of money will be saved and invested which would be spent if the only return offered were the two per cent. of the Post Office savings banks or the four per cent. of the ordinary savings bank.

Prior to the outbreak of the war, a considerable amount of public service bonds bearing a 6% interest rate were being offered. It does not seem unreasonable to expect, therefore, that many concerns may find it now necessary to pay 7% to float bonds and doubtless an even higher rate on short-term loans.

Of course, such a high rate for capital greatly limits the field in which it can be profitably used. It also operates to materially alter the character of engineering design. The engineer to meet present conditions must solve the problem of accomplishing a desired result at the least outlay, even if he considerably increases the cost of maintenance thereby. He will probably have to cope also with high prices of labor, for the advance in the price of foods and the cost of living will inevitably tend to raise wage scales or at least prevent a reduction.

This is in fact a return to the economic conditions which existed in the '70's and early '80's, and the methods used by engineers then will have to be copied by the engineers of today. Fortunately the principal engineering materials, steel, cement, lumber, brick and stone, are not likely to have their cost increased to anything like the figure prevailing forty years or more ago. An exception might be made in the case of lumber; yet at present the stoppage of building work has made lumber more of a drug in the market than it has been for a score of years; and cargoes from the North Pacific coast through the Panama Canal to Atlantic coast markets are likely to keep down lumber prices to a moderate level for many years to come.

There will, therefore, be no need to go so far in economizing material in designing as the engineer of forty years ago went; and the development of labor-saving machinery since that time should enable the engineer nowadays to do far better work at a low labor cost than was possible then.

As to the question when it will be safe to again set in motion the financial machinery of the country, it would be difficult to prophesy. It is evident, however, that the first essential is the restoration of public confidence. With that accomplished, bankers and investors will release funds, which they are now carefully guarding for that the first essential is the restoration of public confiuse in possible emergencies.

Buying and selling of securities are in fact being carried on to a greater or less extent by private arrangement in the manner customary before exchanges were established, but of course, at far greater cost and trouble to both buyer and seller. It may safely be asserted, however, that the stock and other exchanges will be reopened just as soon as restoration of business confidence has proceded to such an extent as to make reopening safe. Those who are responsible for the operation of these exchanges are themselves suffering such heavy financial loss through the closure of the exchanges that they are certain not to postpone reopening a single day beyond the period absolutely necessary.

Letters to the Editor

On the Necessity of Heavier Pavement Foundations

Sir—"The proof of the pudding is in the eating." On this basis, I beg to radically differ from your correspondent who on page 367 of your issue of Aug. 13, makes these statements:

(1) Four- and five-inch foundations are entirely inadequate.
(2) Municipal engineers are extremely shortsighted in continuing to recommend 6 in. as the maximum thickness.
(3) Eight inches on residence streets and 10 and 12 in. on business streets should be required.

Your correspondent's position seems to be based on:

(a) Observation of poor concrete, particularly that made of natural cement, which is no criterion for requirements of sound portland cement concrete.

(b) A statement that we must build for the future and that, as traffic increases, heavier foundations will be required than may be sufficient at the present time. In this he seems to overlook the immense crushing strength of a slab of good sound portland cement concrete having a depth of 4 in. That crushing strength is very much greater than the weight of any load which is likely to ever pass over any road surface, even though the stress on the concrete were not, as it is, very greatly reduced by the wearing surface, say 2 to 6 in. deep, depending on the character of the wearing surface.

In a paper before the American Society of Municipal Improvements at its 1909 Convention at Little Rock, on the subject of "Paving Practice with Respect to Crowns of Roadway Pavements and Concrete Foundations," the writer made the statement that, "under ordinary conditions of well rolled subsoil, 4 in. of first-class portland cement concrete is as good for a pavement foundation as 4 ft., provided the concrete is not disturbed after it is put in place until thoroughly set," and he referred to a then two-year experience as follows:

In the year 1907 the company with which the writer is connected, had a contract for laying "Bitulithic" pavement on several streets, aggregating about 30,000 sq.yd. The specifications provided for foundations of rolled broken stone, poured with bitumen.

On grading the street, a very weak clay subsoil and many recently filled trenches developed, on account of which the city officials permitted a change to 4 in. of portland cement concrete. Before the surface was laid, or the concrete permitted to be otherwise disturbed, it had become thoroughly set, and such that no wagon drawn by horses could break through it. The 12-ton 3-wheel road roller used in compressing the bituminous surface made no depression in the foundation.

Two years later the writer examined the streets and found them to be in perfect condition, with the exception that two poorly filled cross trenches had settled. As a whole, would 4 ft. of concrete have been practically any better than the 4 in. used?

But some engineers will say 4 in. of concrete is not sufficient over poorly filled trenches; to which the answer is, neither is 6 in. nor 8 in.; and the remedy for such cases is, properly backfill trenches and, as an additional precaution, provide an extra 6 in. or even a foot of concrete over and extending a few inches beyond the trench. In other words; put extra concrete where it is needed, i.e., over the trench, and don't on account of the trenches, use several times as much extra concrete over the entire street, on probably nine-tenths of which it is not required.

Based on a wide experience during the intervening five years, he has seen nothing to shake the correctness of his opinion expressed in that paper. The writer goes so far as to say, without fear of contradictory proof, that if a 4-in. portland cement concrete base has failed anywhere, it is due to one of the following causes, none of which are such as to make the general adoption of that depth of concrete undesirable.

(1) An unusually insecure subsoil which is seldom met and which should have been specifically considered by the engineer while making his plans and specifications or while the work was in progress.

(2) Poorly backfilled trenches which should have been guarded against, as they developed in rolling the subgrade, not by placing an inch or two of extra concrete over the whole street where it is not needed, but by perhaps an additional 6 to 12 in. over the weak trenches and extending 6 in. outside the trench on either side. This treatment not only provides a slab of concrete 10 to 16 in. deep, resting on solid earth on either side of the trench, which would probably successfully bridge over the trench and overcome any subsequent settlement, but would generally accomplish that essential end by the use of much less extra concrete than an extra inch or two over the whole street, which would not accomplish the desired end.

(3) Weak street railroad construction, the vibration of which naturally tends to shatter the pavement for a considerable width beyond the ends of the ties. For use with modern heavy interurban and other cars, the concrete from end to end of ties should extend from 6 in. below the bottom of ties to the bottom of the wearing surface, providing, say 18 in. of solid concrete in the railroad area. Then if a heavy rail and closely spaced heavy ties are used, there will be practically no vibration of the track and no damage to the surrounding pavement. If, however, the width between tracks and curb is less than say 16 ft., it is a wise precaution to provide 6 in. concrete for the pavement foundation outside of the railroad area.

(4) Poor concrete, perhaps the most fruitful cause of which is the laying of the pavement surface before the concrete is *thoroughly set*. Under ordinary conditions, I would rather guarantee a pavement surface laid on 4 in. of well set concrete than on 6 or even 8 in. of concrete as green as that which the public, and often engineers who should know better, demand, or the contractors want to cover, so as to get the street "finished."

While urging greater intelligent economy than is perhaps generally practiced in the laying of concrete pavement foundations, the writer deprecates the quite general, and he believes unwise, tendency toward laying thinner wearing surfaces in order to reduce the first cost. The only office of the foundation is to prevent either settlement, vibration or shattering of the foundation by the stress of vehicles passing over the pavement surface. It

has been proved that under ordinary circumstances, that result can be accomplished by either a 4-in. continuous slab of good sound portland cement concrete or by a sound macadam foundation. It is also well known that good portland cement concrete sets harder and harder with age, and I believe the increased stress of increased traffic is nearly nil.

The wearing surface, on the other hand, takes the wear, and, whatever the character of surface, more or less abrasion exists; and it rapidly increases with increased volume of traffic. When any given pavement surface is worn to less than a certain depth the surface is then too thin to sustain the weight and other conditions of traffic.

The difference in labor cost and general expense items connected with the laying of any wearing surface to a depth say one inch more or less than any given standard, is very slight, so that practically the only difference in cost is that of the surface materials.

Against this is what may be termed the "factor of wear" in connection with a greater or less depth of wearing surface. Suppose for instance, that any given wearing surface will not sustain the weight of traffic after its depth is reduced to say 1 in., then if the surface as originally laid has a depth of 2 in., the "factor of wear" is 1 in. If, however, the same character of surface is originally laid to a depth of say 1½ in., then the "factor of wear" is reduced to ½ in.—a decrease in durability of 50% in order to effect a saving in first cost of possibly 20%.

In the case of monolithic bituminous surfaces another point should be considered; to wit, that if the depth of wearing surface as laid is materially more than 2 in. after compression, there is grave danger of not getting perfect compression and the danger from that point outweighs the advantage of increased depth.

In conclusion, in the writer's judgment, true economy demands:

(1) Provide a solid foundation, but not to an unnecessary depth.

(2) Don't "skimp" in depth or in the quality of wearing surface in order to reduce the first cost. This in the end will prove to be poor economy even though the first cost is reduced.

GEO. C. WARREN.

50 Temple Place, Boston, Mass.

Boat-Design Difficulties on the New York State Barge Canal

Sir: In your issue of Aug. 27, 1914, p. 469, under the caption "Type of Boats for the New York State Barge Canal," you reprint an official statement on this subject emanating from State Engineer and Surveyor John A. Bensel. No one will question Mr. Bensel's authority on all subjects dealing with the design and construction of the canal, but when he trenches on the province of the naval architect, he shows himself rather "at sea," as one might expect. I particularly object to his statement:

The shipbuilders have found it impracticable to design a steamer that will satisfy the requirements of the Federal Government for steam craft navigating the Great Lakes and also to keep within the limitations imposed by the Barge Canal, such as restricted clearance of 15½ ft. under bridges and other fixed structures, at high navigable stage, the narrow width of 75 ft. in land sections and shallow depth of 12 ft.

At this stage the shipbuilder and naval architect will want to ask why, if it appeared desirable to the Canal Board that the vessels trading through the canal should also traverse the Great Lakes, were these vexatious limitations Mr. Bensel refers to allowed to be imposed in the construction of the canal.

It is surely self-evident that the New York State Barge Canal must compete with the St. Lawrence system of canals which connect the Great Lakes with the head of ocean navigation at Montreal and must always be handicapped by the fact that the St. Lawrence route does permit of the passage of a combined lake and canal freighter.

The St. Lawrence Canal Locks, in fact, permit vessels of 250-ft. length by 43-ft. beam by 14-ft. draft to pass through, a size of vessel that can go anywhere on any sea in the world and though, owing to rather special construction and peculiar conditions in Canadian navigation, these vessels do not, as a rule, leave Canadian waters, they do trade on the Atlantic coast and go to Europe occasionally and have no difficulty in going anywhere. Now the really extraordinary thing is that if the new Barge Canal locks are only 14 ft. deep, they could take much larger vessels than the Canadian locks, say 300-ft. length by 44-ft. beam easily, which, at the 14-ft. draft of Canadian Canal, would carry over 3000 tons dead weight and could steam all over the Great Lakes and, in fact, all over the globe. In spite of this, by some extraordinary oversight, the bridges on the New York State Barge Canal give a clearance of only 15½ ft., a perfectly absurd dimension for any suitable type of craft with sea-going features, and the earth sections with a bottom width of 75 ft. are equally unsuitable for this ideal ship for which the canal ought to have been designed.

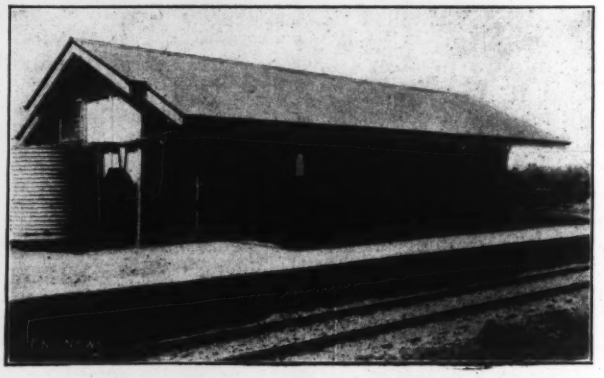
It is quite evident from his other remarks on the best types for use in the New York State Barge Canal that Mr. Bensel does not realize what happens when you attempt to canal a vessel 250 ft. by 40 ft. beam through a section of 75-ft. width with curves the same width as the tangents. A little experience in this line will show that in the least cross wind the vessel, possibly running light, will never be off one bank or the other, that two boats will inevitably collide when attempting to pass, and that to tie one or other of them up to allow the other to pass is to reduce the capacity of the canal to a very serious extent.

It has always seemed extraordinary to naval architects and shipbuilders who have looked into this thing that not the slightest attempt seems to have been made in designing the canal, to decide beforehand what types of boats would be the most suitable and the most likely to be built for use thereon.

Mr. Bensel refers to a solution of this problem by using strings of large steel barges, small tugs being used in the canal and large tugs out on the Great Lakes. It would be interesting to have Mr. Bensel state in what way barges for Great Lake work do not suffer from the restrictions which he states preclude the possibility of shipbuilders producing a combination steamer for canal and Lake work.

The fact is that long experience with the design and operation of Canadian vessels for use in the St. Lawrence Canal and Great Lakes trade have convinced Canadian owners that the tow barge in any form is a failure when competing with the full-sized, full-powered cargo boat

FIGS. 1 AND 2. SHELTER SHEDS ON QUEENSLAND RAILWAYS, AUSTRALIA, OF UNIT SYSTEM REINFORCED CONCRETE



that can take care of herself under all circumstances. This refers to vessels for the handling of the great staple cargoes of grain, coal, pulpwood, package freight and similar cargoes which will constitute the principal traffic for the New York State Canal. In my opinion the only solution now left to the naval architect under the extraordinary conditions imposed upon him is to devise a steel hull with fair shipshape form, large rudder, and biggest possible engine and propeller which will travel absolutely independently, be able to negotiate all bends without touching the banks or tying up, and be seaworthy enough to go round from port to port on Lakes Erie and Ontario without running the risk of disaster. It can be done, but the dimensions will be very far short of anything which Mr. Bensel has indicated, and very inferior in size and capacity to the competing Canadian canal types and for that most unfortunate handicap the users of the New York State Barge Canal will have to hold the naval architect and the shipbuilder free from blame.

JOHN REID.

17 Battery Pl., New York City, Aug. 28, 1914.

Reinforced-Concrete Shelter Sheds on an Australian Railway

Sir—I was much interested in reading the article on p. 737, of the issue of Apr. 2, 1914, of ENGINEERING NEWS, describing the adoption of reinforced concrete for construction of shelter sheds. A similar method of construction was adopted by the Railway Department of Queensland, Australia, four or five years ago, and I give herewith views of our buildings, thinking they may be of interest.

Upon the review being made by the Engineer for Construction of the great expenditure involved in the maintenance branch of the railway for the upkeep of the wooden structures used for shelter-shed accommodation, and with a desire for a more economical method of construction, it was decided to erect in place of the wooden buildings, reinforced-concrete structures.

Upon such a decision being made, designs were prepared on the unit system with a view to facilitating erection and carrying out extensions in the quickest manner with a minimum of labor.

A general description of the method adopted, together with the accompanying view, will readily convey to the professional mind the general principles thereof.

Piers or dwarf walls are erected under the building for the reception of the superstructure, according to the requirements of the site. Beams reinforced with ½-in. rods are then placed in position. These piers are holed to receive the reinforced verticals of the side and end wall. The verticals are rabetted on each side for the reception of 4-ft. by 2x18-in. wall slabs, rough cast on the outside, reinforced with netting; the reinforcing rod at the top of the vertical being extended to such length as required to receive the wooden plate placed for the reception of the rafters of the roof, and the rods at the

bottom of the vertical being bedded in the floor beams.

The floor is of concrete 4 in. thick, reinforced with rods as required, and made in 4-ft. squares. The roof is constructed of wood and covered with tile, giving the building a neat and serviceable appearance.

The cost does not exceed that of a wooden structure of corresponding size.

The unit adopted is 4 ft. for all work, that length being a measure common to all designs in our country stations. By this means additions to or deductions from can be easily effected.

R. E. SEXTON,
Engineer in Charge of Construction, Queensland Railways.
Brisbane, Queensland, June 8, 1914.

NOTES AND QUERIES

In "Engineering News," July 9, 1914, p. 72, it was stated that the puddle core wall of the Throttle dam in Raton, N. M., was faced with No. 12 gage corrugated galvanized "Ingot Iron." We have since learned that what was used was No. 20 gage corrugated painted "Ingot Iron."

Locomotive Cab Signals and Audible Track Signals in Europe

Cab signals (audible or visual) on locomotives, as auxiliary to fixed signals beside the track, have been tried only to a limited extent in this country, but have been applied extensively in Europe for the past 40 or 50 years. At the meeting of the Institution of Mechanical Engineers, of Great Britain, at Paris, in July, papers were read describing the use of cab signals on five French railways. In all cases the contact is made by a wire brush on the engine on a contact bar placed between and parallel with the rail. From the papers noted above we have compiled the following particulars as to practice on French railways.

NORTHERN RAILWAY—This road has used such signals for some 40 years. With the present equipment the contact bar or track device is set either opposite the signal or at a distance which may be as much as 650 ft. It is an oak beam 4 in. thick, 6 in. wide, and 6½ ft. long, the wood being boiled in oil for insulating and preservative purposes. It is supported on iron blocks, with its top 9½ in. above the ties and about 4 in. above the rails. The entering end is beveled off, and is protected against hanging parts on the trains by a timber 5 ft. long placed in advance of it and having its top inclined. Where trains pass at speeds of over 30 m.p.h. a second contact bar is placed against the end of the first one to insure sufficient duration of contact. The wooden bar is faced with a brass plate (on wood or porcelain washers), having riveted to the underside a square copper block which is connected to the battery wire.

The contact brush on the engine consists of a series of hard but elastic copper wires soldered to a metal plate. From this a cable leads to the electro-magnet operating the cab whistle, while the other wire from the electro-magnet is grounded through the wheels and rails. The warning apparatus has a whistle with a lever which carries a plunger valve, working in a tube connecting the whistle with the boiler or a compressed-air chamber. The lever is pulled down by a rod having a heavy spring which tends to hold the valve open, but this rod is held up normally by its lower end resting on the arm of an electro-magnet. When a current is sent through the coils of this, the magnet ceases to attract the armature, so that the arm drops and the lever is pulled down by the spring, opening the valve and causing the whistle to sound until the engineman restores the apparatus to normal position by pressing a lever.

The signal is a disk on a tall rod, which revolves on its vertical axis, and the disk has a switch operated by a finger keyed to the post which supports the rod. When the disk has revolved 70° from the "clear" position the switch makes the circuit with the track contact mechanism. The same switch operates a bell at the station to indicate the position of the signal. For the track contact there is a 12-cell battery, with its positive pole connected to the brass contact plate and its negative connected to the switch. For the station bell there is an eight-cell battery, both batteries being placed in a small concrete chamber at the foot of the signal post.

In connection with the device described there is an indicator giving visual and audible warning at the station and signal cabin when a train passes the distant signal. This has two track contacts, the first sounding the cab whistle and the second sounding a bell at the station or signal cabin; this operates for every train (or engine), regardless of the position of the signal. The apparatus in the signal cabin is a vibrating bell and a miniature blade or disk.

EASTERN RAILWAY—The system is in general similar to that of the Northern Ry., but two track contacts are used, one near the signal and the other at a distance which may be as great as 1300 ft., so that if the signal is at "stop" the engineman has an advance warning and then a second warning at the signal. The wood bar, however, has been replaced on this road by a "gridiron" of steel bars or angles, in order to eliminate contact failures due to frost.

Controlling apparatus (for which various devices have been tried) is used to prevent the engineman from shutting off the sound too quickly. In addition to this, the operation of the cab signal is recorded automatically on the continuous chart of the speed recorder, with which French locomotives are commonly equipped. The apparatus sounds a bell when the engine passes a "clear" signal, thus giving the engineman a positive indication for either position of the signal.

SOUTHERN RAILWAY—The track contact apparatus is about 12 ft. long, and the battery is on the engine instead of on the ground. A steam whistle is used, and there is both warning and recording apparatus. When an engineman sees a "stop" signal, he shifts the handle of the control apparatus and thus causes a mark to be made on the speed-indicator chart, the mark showing whether or not he has operated the device in proper time. On reading the signal, the brush on the engine makes contact with the track device, and the circuit is closed, the current operating the valve of the whistle device and two special pens in the speed indicator. The engineman can shut off the whistle device by closing the valve. This road has 20 locomotives and 30 automatic signals equipped in this way.

STATE RAILWAYS—Four cab-signal devices are being tried on this system: The Cousin (55 engines), the Van Braan (28), the Eastern Ry. (27), and the Augereau (2 engines).

PARIS, LYONS & MEDITERRANEAN RY.—This road is making experiments with 140 engines on 258 miles of line having 316 disk signals. Electricity is used only to close the circuit, all operations being done mechanically. There is an automatic device for releasing the horn and discharging compressed air through it, and another device for marking the speed-indicator chart. On engines having air brakes, the air is taken from the brake receiver; otherwise a charged receiver is placed on the engine.

PARIS & ORLEANS RY.—For over 50 years this road has employed apparatus for placing torpedoes and giving an audible signal besides the track instead of in the cab. The torpedoes are placed when the signal is set at the "stop" position, and severe punishment is prescribed for men who pass a "stop" signal. The signal is set at such a distance from the point to be protected as to enable a train to be stopped before reaching it, even if the engineman does not shut off steam and apply the brakes until he hears the torpedoes. The sound is loud enough to be heard by the trainmen and passengers. The system is considered by Mr. Solacroup (Superintendent of Motive Power) to be preferable to a cab signal, and the value of its moral effect is seen in the fact that there is no record of any accident having occurred due to a train running past a "stop" signal.

A Machine Plowing Record is reported from Fremont, Neb., where a gasoline engine "caterpillar" traction engine pulled a gang of 24 plows and turned 28 ft. wide, 7 to 8 in. deep, at 2⅛ miles per hour, equivalent to about 7¼ acres per hour. The tractor was a Holt type, with six 7½x8-in. cylinders, developing 130 b.hp.; the caterpillar track consisted of an endless chain of treads on each side, 2 ft. wide and 10 ft. in circumference. The plow was made up of four six-base Oliver units handled with one hitch.

Putting a Concrete Lining, Later a New Skin, on an Old Hull

By Horace M. Marshall*

The general deterioration of the steel hull of a steam launch after 13 years' service, demanded unusual methods to save the boat. The first protective measure was to line the hull with cinder concrete reinforced with chicken wire. This served for several years when the shell was so thin that it was feared the concrete would drop out. A new steel skin was then put on.

The hull was 65½ ft. long, 10½-ft. beam, and was 4 ft. deep amidships; the plating was of soft steel, 4⅓ lb. per sq.ft. The draft was 20 to 24 in., according to load, and the wheel 30 in. in diameter. The wheel revolves in a tunnel, and when the boat is not under way only about two-thirds is below the water line; but it is submerged while running, the water rises and fills the tunnel.

The boat was originally driven by a triple-expansion direct-connected steam engine which developed 90 to 100 hp. at 500 r.p.m., and attained a speed of 12.6 mi. per hr. in still water. Later a quadruple-expansion condensing engine was installed, which, though it developed 125 hp., increased the speed only to about 13 mi. per hr. At 50 hp., the speed was 11½ mi. per hr.

The boat was built in 1897 at a cost of $10,000 and purchased by the United States government in 1902 for half that amount, for purposes of inspection and light towing on shallow rivers.

Fuel consumption was small (about 180 lb. of coal per hr.), costing 40 cents. Storage capacity, however, was only about three tons, which limited travel between coal piles to about 30 hours. The crew comprised a pilot, engineer and fireman.

In 1909, the engine and boiler were removed and a 50-hp. four-cylinder, four-cycle gasoline engine installed. The gasoline consumption is 6 gal. per hour. The storage capacity is 420 gal. A new pilot house was constructed suspended half in the engine room and half extending above the roof, so as to give pilot-house control and enable one man to run the boat.

Fig. 1. Sinking Dry-Dock; Boat Floating; Water Filling Shell
Fig. 3. New Skin Applied

Fig. 2. Raising Dry-Dock; Boat Lowering into Shell
Fig. 4. Boat with New Skin Leaving Dry-Dock

When the boat was 13 years old, "pinholes" developed all over the plating and the plates were so rusted that it became an every-day occurrence to knock a hole in the bottom during low water and sink the boat.

This state of affairs was met by a concrete lining. The hull, inside, was thoroughly scraped and cleaned. Chicken wire was spread over the ribs, and cinder concrete was rammed down through the mesh all over the bottom of the boat, extending well up on the sides above the waterline. The lining was from 4 to 5 in. thick at the keel and thinned toward the sides of the hull. The added weight was 8000 lb., which increased the draught 3 to 6

*U. S. Engineer Office, Vicksburg, Miss.

in. The boat now worked satisfactorily for three or four years when the steel plates had grown so thin that it was feared the concrete would drop out through the bottom. In other respects, the boat was in good condition.

In dry dock, templates were made to fit the bottom, at intervals of 2 ft. From these a timber frame was built with ribs of 2x4-in. pine. Metal plates of No. 10 gage were bolted transversely on the outside of this frame, with seams lapped and riveted, with thin tar paper between the lap of the metal plates. The timber frame was then removed.

The dry dock was flooded, the boat floated over the shell, and the dock was raised. Guide posts placed the boat accurately above the shell as the dock came up. Melted paving pitch was poured in the space between the hull and the new skin. A 2x4-in. wood nosing strip running around the top of the new construction was bolted on through the shell and hull. About 7000 lb. of concrete were taken out of the hull, leaving only the concrete which was under the engine bed. After the work was finished, the draft was 18 in. forward and 27 in. aft.

The shell cost $419, and the labor of constructing it about the same amount. It is expected that the new skin will last 12 or 15 years. A new hull would have cost from $3000 to $4000.

The accompanying photographs show the boat at several stages of the work.

The Transportation of Debris by Running Water*

By Grove Karl Gilbert

Scope—The finer débris transported by a stream is borne in suspension. The coarser is swept along the channel bed. The suspended load is readily sampled and estimated, and much is known as to its quantity. The bed load is inaccessible and we are without definite information as to its amount. The primary purpose of the investigation was to learn the laws which control the movement of bed load, and especially to determine how the quantity of load is related to the stream's slope and discharge and to the degree of comminution of the débris.

Method.—To this end a laboratory was equipped at Berkeley, Calif., and experiments were performed in which each of the three conditions mentioned was separately varied and the resulting variations of load were observed and measured. Sand and gravel were sorted by sieves into grades of uniform size. Determinate discharges were used. In each experiment a specific load was fed to a stream of specific width and discharge, and measurement was made of the slope to which the stream automatically adjusted its bed so as to enable the current to transport the load.

The Slope Factor—For each combination of discharge, width, and grade of débris there is a slope, called competent slope, which limits transportation. With lower slopes there is no load, or the stream has no capacity† for load. With higher slopes capacity exists; and increase of slope gives increase of capacity. The value of capacity is approximately proportional to a power of the excess of slope above constant slope. If S equal the stream's slope and σ equal competent slope, then the stream's capacity varies as $(S - \sigma)^n$. This is not a deductive, but an empiric law. The exponent n has not a fixed value, but an indefinite series of values depending on conditions. Its range of values in the experience of the laboratory is from 0.93 to 2.37, the values being greater as the discharges are smaller or the débris is coarser.

The Discharge Factor—For each combination of width, slope and grade of débris there is a competent discharge k. Calling the stream's discharge Q, the stream's capacity varies as $(Q - k)^o$. The observed range of values for o is from 0.81 to 1.24, the values being greater as the slopes are smaller or the débris is coarser. Under like conditions o is less than n; or, in other words, capacity is less sensitive to changes of discharge than to changes of slope.

The Fineness Factor—For each combination of width, slope and discharge, there is a limiting fineness of débris below which no transportation takes place. Calling fineness (or degree of comminution) F and competent fineness ϕ, the stream's capacity varies $(F - \phi)^p$. The observed range of values for p is from 0.50 to 0.62, the values being greater as slopes and discharges are smaller. Capacity is less sensitive to changes in fineness of débris than to changes in discharge or slope.

The Form Factor—Most of the experiments were with straight channels. A few with crooked channels yielded nearly the same estimates of capacity. The ratio of depth to width is a more important factor. For any combination of slope, discharge and fineness, it is possible to reduce capacity to zero by making the stream very wide and shallow or very narrow and deep. Between these extremes is a particular ratio of depth to width ρ, corresponding to a maximum capacity. The values of ρ range, under laboratory conditions, from 0.5 to 0.04, being greater as slope, discharge and fineness are less.

Velocity—The velocity which determines capacity for bed load is that near the stream's bed, but attempts to measure bed velocity were not successful. Mean velocity was measured instead. To make a definite comparison between capacity and mean velocity it is necessary to postulate constancy in some accessory condition. If slope be the constant, in which case velocity changes with discharge, capacity varies on the average with the 3.2 power of velocity. If discharge be the constant, in which case velocity changes with slope, capacity varies on the average with the fourth power of velocity. If depth be the constant, in which case velocity changes with simultaneous changes of slope and discharge, capacity varies on the average with the 3.7 power of velocity. The power expressing the sensitiveness of capacity to changes of mean velocity has in each case a wide range of values, being greater as slope, discharge and fineness are less.

Mixtures—In general, débris composed of particles of a single size is moved less freely than débris containing particles of many sizes. If fine material be added to coarse, not only is the total load increased but a greater quantity of the coarse material is carried.

Modes of Transportation; Movement of Particles—Some particles of the bed load slide; many roll; the multitude make short skips or leaps, the process being

*Author's prefatory abstract of Professional Paper 86, U. S. Geological Survey, Washington, D. C. The paper itself is "based on an experiment made with the assistance of Edward C. Murphy," at the University of California.

†Capacity is defined for the purposes of this paper as the maximum load of a given kind of débris which a given stream can transport.

called saltation. Saltation grades into suspension. When particles of many sizes are moved together the larger ones are rolled.

MODES OF TRANSPORTATION; COLLECTIVE MOVEMENT—When the conditions are such that the bed load is small, the bed is molded into hills, called dunes, which travel downstream. Their mode of advance is like that of eolian dunes, the current eroding their upstream faces and depositing the eroded material on the downstream faces. With any progressive change of conditions tending to increase the load, the dunes eventually disappear and the débris surface becomes smooth. The smooth phase is in turn succeeded by a second rhythmic phase, in which a system of hills travel upstream. These are called antidunes, and their movement is accomplished by erosion on the downstream face and deposition on the upstream face. Both rhythms of débris movement are initiated by rythms of water movement.

APPLICATION OF FORMULAS—While the principles discovered in the laboratory are necessarily involved in the work of rivers, the laboratory formulas are not immediately available for the discussion of river problems. Being both empiric and complex, they will not bear extensive extrapolation. Under some circumstances they may be used to compare the work of one stream with that of another stream of the same type, but they do not permit an estimate of a river's capacity to be based on the determined capacities of laboratory streams. The investigation made an advance in the direction of its primary goal, but the goal was not reached.

LOAD VERSUS ENERGY—The energy of a stream is measured by the product of its discharge (mass per unit time), its slope, and the acceleration of gravity. In a stream without load the energy is expended in flow resistances, which are greater as velocity and viscosity are greater. Load, including that carried in suspension and that dragged along the bed, affects the energy in three ways: (1) It adds its mass to the mass of the water and increases the stock of energy *pro rata*. (2) Its transportation involves mechanical work, and that work is at the expense of the stream's energy. (3) Its presence restricts the mobility of the water, in effect increasing its viscosity, and thus consumes energy. For the finest elements of load the third factor is more important than the second; for coarser elements the second is the more important. For each element the second and third together exceed the first, so that the net result is a tax on the stream's energy. Each element of load, by drawing on the supply of energy, reduces velocity and thus reduces capacity for all parts of the load. This principle affords a condition by which total capacity is limited. Subject to this condition a stream's load at any time is determined by the supply of débris and the fineness of the available kinds.

FLUME TRANSPORTATION—In the experiments described above—experiments illustrating stream transportation—the load traversed a plastic bed composed of its own material. Other experiments were arranged in which the load traversed a rigid bed, the bottom of a flume. Capacities are notably larger for flume transportation than for stream transportation, and their laws of variation are different. Rolling is an important mode of progression. For rolled particles the capacity increases with coarseness, for leaping particles with fineness. Capacity increases with slope and usually with discharge also, but the rates of increase are less than in stream transportation. Capacity is reduced by roughness of bed.

VERTICAL VELOCITY CURVE—The vertical distribution of velocities in a current is controlled by conditions. The level of maximum velocity may have any position in the upper three-fourths of the current. In loaded streams its position is higher as the load is greater. In unloaded streams its position is higher as the slope is steeper, as the discharge is greater and as the bed is rougher.

PITOT TUBE—The constant of the Pitot velocity gage—the ratio between the head realized and the theoretic velocity head—is not the same in all parts of a conduit, being less near the water surface and greater near the bottom or side of the conduit.

NEWS NOTES

A Ditching Machine Broke a 6-In. Water Main in Springfield, Ohio, on Aug. 27.

A Broken Water Main in Brooklyn, N. Y., flooded the new Seventh Ave. subway at 21st St. to a depth of 12 ft.

An Elevator Cable broke in a building in Philadelphia, Penn., on Aug. 28, injuring the three occupants of the car.

An Undermined Wall Failed in Toronto, Ont., on Sept. 1, and fell into the adjacent excavation for a large office building. At about 1 p.m., the side wall of the building settled, opening a crack in the front. It was immediately attempted to shore up the wall, but at 8 p.m. half of it fell.

Floods in Manila, P. I., followed a week's heavy rain. During the 48 hours ending at midnight, Sept. 1, the precipitation was 16.4 in. A message from Governor-General Harrison states that the lowest sections of Manila were flooded to a depth of 3 to 5 ft. Eight persons were drowned.

A Survey of the Street Trees of the Borough of Manhattan, New York City, is being made by the Landscape Engineer of the State College of Forestry at Syracuse. In one ward only 200 shade trees were found on 12 mi. of residential streets, or one to 440 ft.

Magnesite from California will take the place in the United States of that formerly imported from Austria, but now held up by the European War. A fleet of Jeffrey Quad motor trucks is already hauling the American product to Tidewater, Calif., for shipment through the Panama Canal.

Disbursements for Rapid Transit Subway Work in New York City from the beginning of the existing system to June 30, 1914, aggregate $100,338,171.45 of which some $48,000,000 is represented by the Interborough system now running; the remainder has been spent on the dual system under way. These figures are exclusive of real estate purchases. The total of contracts awarded for the new lines aggregated $135,910,269 on Aug. 15.

The First Liquefield Chlorine Gas Plant for Disinfecting a Municipal Water-Supply in Massachusetts was put into use on Sept. 3 at Wakefield. The town takes its supply from Crystal Lake, which, while providing a clear, sparkling water, is liable to occasional pollution. The use of liquefied chlorine gas as a sterilizing agent was recommended by Edward C. Sherman, Consulting Engineer, of Boston, who is supervising the operation of the plant which was installed by the Electro-Bleaching Gas Co., of New York. It has been under test for about six weeks and has given satisfactory results, reducing the bacterial content to very low counts. Charles A. Dean is Chairman of the Wakefield Water Board.

Protection of Pedestrians in front of a garage which motor vehicles enter and leave by crossing the sidewalk, was recently accomplished in Louisville, Ky., by installing a warning lamp, on each side of whose globe is the word "Danger," and by connecting up a bell which operates automatically whenever a vehicle approaches the exit door. The lamp is lit from sunset to sunrise. This safety device was installed by the proprietor of the garage on his own initiative. The Society for Electrical Development believes it would be worth while for local authorities everywhere to consider the enforcement of such protection wherever such a menace exists.

American Production of Industrial Materials, formerly imported but now cut off by European war, is being sought by

the U. S. Department of the Interior. Secretary F. K. Lane, on Sept. 3, called a conference of manufacturers of coal-tar, pharmaceutical and general chemical products to find out in what ways the Government could aid in supplying the present deficiency. The normal importation of coal-tar dyes alone amounts to $7,000,000 per year. There were some 25 representatives present. It was definitely suggested (1) that there should be a system instituted in this country similar to the one in England, whereby if a foreign patent is not used and manufactured within a reasonable time, it will be opened to use by American manufacturers, and (2) that a law should be passed whereby unfair competition will be cut off. This last would preclude the practice which it is alleged is now followed of selling foreign products below the costs of manufacture. It was further suggested (3) that the secretary use his efforts through the State Department to secure permission for the exportation of dyestuffs from Germany, and (4) that the wisest way to develop a great coal tar product industry in the United States was to establish a government plant whose primary purpose should be the making of nitro derivatives of coal tar and other materials needed for explosives by the War and Navy Departments. The startling statement was made that the United States today was absolutely dependent on Germany for some of the essential elements in smokeless powder, and that if Germany stopped their manufacture or prohibited export to his country, America would be helpless.

It is reported that the experimental potash plant at Searles Lake, Calif., will be arranged for continuous production on as nearly commercial basis as possible. The initial output will be five tons per day, but a plant is under construction to turn out 120 tons.

PERSONALS

Mr. Frank C. Miller has been appointed City Engineer of Sacramento, Calif., succeeding Mr. Albert Givon, Assoc. M. Am. Soc. C. E., resigned.

Mr. C. M. Buck, Assoc. M. Am. Soc. C. E., of Guthrie, Okla., has been appointed Division Engineer of the Atchison, Topeka & Santa Fé Ry., with office at Emporia, Kan., succeeding Mr. F. L. Guy, resigned.

Mr. W. D. Warren, formerly Division Engineer of the New York, New Haven & Hartford, Conn., has been transferred to Providence, R. I., to succeed Mr. W. T. Spencer, Assoc. M. Am. Soc. C. E., promoted.

Prof. John C. Olson, formerly of the Polytechnic Institute of Brooklyn, N. Y., has been appointed Professor of Chemistry and head of the department of chemistry at Cooper Union, New York City.

Mr. W. J. Miller, Master Mechanic of the St. Louis Southwestern Ry. of Texas, at Tyler, Tex., has been promoted to be Superintendent of Motive Power of the company with headquarters at Pine Bluff, Ark., succeeding the late T. E. Adams.

Mr. Harrison P. Eddy, M. Am. Soc. C. E., of Metcalf & Eddy, Consulting Engineers, Boston, Mass., has been appointed Consulting Engineer of the city of Dayton, Ohio, in connection with the improvement of the sewerage system of that place.

Mr. Jamees N. Hatch, M. Am. Soc. C. E., Structural Engineer for Sargent & Lundy, Consulting Engineers, Chicago, Ill., has resigned to open an officee for private practice at 1525 Old Colony Bldg., Chicago. He intends to specialize in appraisal and valuation work.

Mr. H. E. Kellenberger has been appointed Senior Signal Engineer of the Central district of the Division of Valuation, Interstate Commerce Commission, with headquarters at Chicago, Ill. He was formerly Superintendent of Signals on the Chicago Great Western Railway.

Mr. W. J. Gough, Assoc. M. Am. Soc. C. E., formerly Assistant Engineer of the Coronado Beach Co., San Diego, Calif., has been elected Vice-President and Superintendent of the Los Angeles & San Diego Beach Ry., succeeding Mr. R. B. Talbot, resigned, to engage in other business.

Mr. C. F. Yardley, Track Supervisor of the New York, New Haven & Hartford R.R., at Providence, R. I., has been promoted to be Division Engineer of the Central New England R.R., at Hartford, Conn., to succeed Mr. H. E. Astley, promoted to be Division Engineer of the New York, New Haven & Hartford R.R., at Hartford, Conn.

Mr. Harry R. Kurrie, General Attorney of the Chicago, Indianapolis & Louisville Ry., has been elected President to succeed Mr. F. A. Delano, M. Am. Soc. C. E., resigned, as noted in our issue of Aug. 20. Mr. Kurrie is 39 years old and entered the service of the Chicago, Indianapolis & Louisville Ry. in 1902 as Assistant General Solicitor.

Mr. Charles Saville, Assoc. M. Am. Soc. C. E., has resigned from the firm of Hering & Gregory, Consulting Engineers, New York City, and will spend the coming winter in Boston, Mass., taking a graduate course at the School for Health Officers at Harvard University and the Massachusetts Institute of Technology. His home address is Waban, Mass.

Mr. J. A. Lindstrand has been appointed Senior Architect in the Central district of the Division of Valuation, Interstate Commerce Commission, with headquarters at Chicago, Ill. Mr. Lindstrand was for fifteen years in the Architect's office of the Chicago, Burlington & Quincy R.R. and for the past eight years has been Architect for the Chicago, Milwaukee & St. Paul Ry.

Mr. A. H. Whiteside, former Manager of the power and electric department of the Allis-Chalmers Co., Milwaukee, Wis., has been appointed Vice-President and General Sales Manager of the Goulds Manufacturing Co., of Seneca Falls, N. Y. Mr. Whiteside succeeds Mr. W. E. Davis, who has been obliged to give up some of his active duties due to injuries sustained several months ago.

Mr. Fred M. Baumgardner has been appointed Senior Inspector of Motive Power in the Central district of the Division of Valuation, Interstate Commerce Commission, with headquarters at Chicago, Ill. His experience has been with the Mechanical Department of the Union Pacific R.R. and Illinois Central R.R., having served as Round House Foreman, General Foreman and Masted Mechanic on the latter road and having recently been made Master Mechanic at Clinton, Ill.

Mr. Fred B. Corey, F. Am. Inst. E. E., for 11 years in the engineering department of the General Electric Co., Schenectady, N. Y., and for the past three years Engineer of Inspection and Tests of the Union Switch & Signal Co., Swissdale, Penn., reporting to the General Manager, has resigned that office to open an office at 404 Arrott Bldg., Pittsburgh, Penn., as Consulting Engineer, specializing in inspection methods and organizations for manufacturing companies. He will also specialize in electric railway signaling and allied subjects.

Mr. D. K. Van Ingen has been appointed Second Assistant Field Engineer in the Central district of the Division of Valuation, Interstate Commerce Commission, with headquarters at Chicago, Ill. He was educated at Rensselaer Polytechnic Institute, and has been with the Chicago & Northwestern Ry., Choctaw, Oklahoma & Gulf R.R. and New York Central & Hudson River R.R., having been Assistant Engineer in charge of valuation work on yards and terminals in Iowa on the Chicago & Northwestern since 1913, and within the last month he was appointed Chief Draftsman in the valuation department of the same road.

Mr. W. E. Van Hook has been appointed Office Engineer for the Central district of the Division of Valuation, Interstate Commerce Commission, with headquarters at Chicago, Ill. He is a graduate of the University of Wisconsin. The first five years after graduation he was on railroad work in location, construction, maintenance and valuation work with the Chicago & Northwestern Ry., Chicago & Alton R.R., and the Chicago, Milwaukee & Puget Sound R.R. For the past three years he has been with the Illinois Railroad & Warehouse Commission and later the Public Utilities Commission of the State of Illinois.

Mr. Truman P. Gaylord, Assoc. Am. Inst. E. E., District Manager of the Westinghouse Electric & Manufacturing Co., at Chicago, Ill., has been elected Acting Vice-President to succeed Mr. H. D. Shute, promoted, as noted elsewhere. Mr. Gaylord graduated in electrical engineering from the University of Michigan in 1895. Until he joined the staff of the Westinghouse Electric & Manufacturing Co. in 1899, Mr. Gayford was Assistant Professor of electrical engineering at the Armour Institute of Technology and with the Chicago Edison Co. He was appointed District Manager of the Chicago office of the Westinghouse Co. in 1902.

Mr. B. B. Kelliher, M. Can. Soc. C. E., has resigned as Chief Engineer of the Grand Trunk Pacific Ry. on account of ill health. Mr. Kelliher was born in Ireland in 1862 and graduated from Dublin University. He served an apprenticeship in civil engineering in his native country and in 1886 came to the United States, where for four years he was with the engineering corps of the Union Pacific R.R. For 14 years he was a railway and mining engineer in the Northwest and in the Rocky Mountain States. He joined the staff of the Grand Trunk Pacific Ry. in 1904 as Division Engineer at Winnipeg, Man., and a year later was made Chief Engineer.

Mr. W. A. Christian, M. Am. Soc. C. E., formerly First Assistant to the Chief Engineer of the Chicago Great Western R.R., has been appointed by the Interstate Commerce Commission Senior Civil Engineer in charge of the roadway and track department for the Central district of the Division of Valuation, with headquarters at Chicago, Ill. Mr. Christian graduated from the University of Cincinnati and his early work was with the Baltimore & Ohio, Chesapeake & Ohio, and C. C. C. & St. L. railroads. In 1894 he was Assistant County Engineer of Hamilton County, Ohio, and from 1895 to 1902 was with the United States Army Engineer Corps as U. S. Assistant Engineer in charge of plans and estimates for river and harbor improvements. He was then with the Big Four R.R. for about a year on maintenance work, and from 1903 to 1907 was again with the U. S. Army Engineer Corps as U. S. Assistant Engineer in charge of plans and estimates for concrete movable dams in the Ohio River. Since 1907 he has been with the Chicago Great Western R.R. as Assistant Chief Engineer.

Mr. Henry D. Shute, Assoc. Am. Inst. E. E., Acting Vice-President of the Westinghouse Electric & Manufacturing Co., Pittsburgh, Penn., has been elected Treasurer to succeed Mr. T. W. Siemon, who recently resigned to become Secretary-Treasurer of the Union Switch & Signal Co., Swissvale, Penn. Mr. Shute is an electrical engineering graduate of the Massachusetts Institute of Technology, class of 1892, and in 1893 entered the employ of the Westinghouse Electric & Manufacturing Co. as an apprentice. He entered the engineering department of the company in 1897 and a year later was transferred to the commercial department. In 1901 Mr. Shute was promoted to be the head of the alternating current division of the commercial department, and two years later was made Assistant to the Vice-President, in which position he had much to do with the development of heavy electric traction and single-phase railway work. He has been Acting Vice-President since 1910.

OBITUARY

William A. Stern, senior member of the firm of Stern & Silverman, which controls many street and electric railway properties and holds a franchise for a tunnel under the Delaware River at Philadiphia, died Sept. 4, at his home in Philadelphia. He was 54 years old.

Jacob H. Olhausen, former General Superintendent of the Central R.R. of New Jersey, died at his home in New York City, Sept. 2, aged 76 years. He entered the service of the Philadelphia & Reading Ry. as a water boy. He was successively promoted to be brakeman, conductor, and eventually Division Superintendent.

Henry Harbinson Sinclair, M. Am. Inst. E. E., a promineent electrical engineer of Los Angeles, Calif., died Aug. 31, at his home in Pasadena. He was born in Brooklyn, N. Y., 57 years ago. He went to California in 1889 and became connected with several large hydro-electric projects. He acted as consulting expert in the organization of the Great Western Power Co., of San Francisco, Calif., of which he later served as Vice-President. He was a well known yachtsman and twice the winner of the ocean race from San Francisco to Honolulu. He is survived by a widow, a son and a daughter.

Moritz George Lippert, for almost half a century connected wth the drafting department of the Phœnix Bridge Co., and its predecessors, Phœnixville, Penn., died at his home in Phœnixville, Aug. 30. More than 300 young engineers and draftsmen have been trained under him as Chief Draftsman, during the many years he had been with the company. Mr. Lippert was born Nov. 19, 1846, the son of physician in Leipsig, Germany. He graduated from the Royal Polytechnic Institute at Dresden and for several years practiced his profession in Germany. In 1869 he came to the United States and settled in Phœnixville, where in the preceding year the firm of Kellogg, Clarke & Co. had opened an office for designing and erecting iron bridges and kindred structures. The firm was reorganized in 1871 as Clarke, Reeves & Co. and 1884 was transformed into the Phœnix Bridge Co. Under the late Adolphus Bonzano as Chief Engineer of Kellogg, Clarke & Co., Mr. Lippert was appointed to take charge of the drafting room, where he continued for many years. Besides his professional work he found time for many public and philanthropic enterprises. For many years he was the moving spirit and director of a town coöperative society for supplying groceries and other commodities. As Secretary of the local board of health he performed a vast amount of work, doing it so thoroughly that his reports became models to be followed by the health authorities of other towns. For many years he served as Secretary of the local hospital board. The local newspaper pays this tribute to his sterling qualities:

No citizens of this country—native or naturalized—ever fulfilled all the duties of citizenship more conscientiously and more intelligently than did Mr. Lippert. Personally he had an equable temper, or under perfect control, a keen sense of justice and a saving sense of humor. He was exacting only with himself, to all others he was most lenient and most forgiving. With the formal courtesy of the old world softened by his contact with the new, he was most companionable and most lovable. He loved the world and the world loved him and will miss him.

Mr. Lippert is survived by a widow, a daughter, two sisters and a brother, F. George Lippert, Assistant Engineer of the Phœnix Bridge Co.

ENGINEERING SOCIETIES

COMING MEETINGS

INTERNATIONAL ASSOCIATION OF MUNICIPAL ELECTRICIANS.
Sept. 15-18. Convention in Atlantic City, N. J. Secy., Clayton W. Pike, Electrical Bureau, Philadelphia, Penn.

ILLUMINATING ENGINEERING SOCIETY.
Sept. 21-25. Annual convention in Cleveland, Ohio. Secy., Joseph D. Israel, 29 W. 39th St., New York City.

RAILWAY SIGNAL ASSOCIATION.
Sept. 22-24. Annual convention in Bluff Point, N. Y. Secy., C. C. Rosenberg, Times Building, Bethlehem, Penn.

ATLANTIC DEEPER WATERWAYS ASSOCIATION.
Sept. 22-26. Annual convention at New York. Secy., Addison Burke, Philadelphia, Penn.

INTERNATIONAL IRRIGATION CONGRESS.
Oct. 5-9. At Calgary, Alta. Secy., Andrew Miller, Industrial Commissioner, Calgary.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Oct. 12-16. Annual convention at Atlantic City, N. J. Secy., E. B. Burritt, 29 West 39th St., New York City.

NATIONAL COUNCIL FOR INDUSTRIAL SAFETY.
Oct. 13-15. Annual "Safety Congress," at Chicago, Ill. Secy., W. H. Cameron, Continental Bank Building, Chicago.

AMERICAN RAILWAY BRIDGE AND BUILDING ASSOCIATION.
Lichty, 319 North Waller Ave., Chicago, Ill.
Oct. 20. Convention at Los Angeles, Calif. Secy., C. A.

INTERNATIONAL ASSOCIATION OF FIRE ENGINEERS.
Oct. 20-23. Annual convention at New Orleans. Secy., James McFall, Roanoke, Va.

AMERICAN ROAD CONGRESS.
Nov. 9-14. Atlanta, Ga. Permanent headquarters, American Highway Association, Colorado Building, Washington, D. C.

International Rubber Congress and Exhibition—Among the international congresses scheduled this year and next is the Rubber Congress set for Sept. 8-Oct. 10, 1914, at Batavia, Java. Although we have not been informed of the abandonment of the Congress in view of the European war, it is quite possible this has been done. An extensive program had been prepared. The local secretary is Dr. A. A. L. Rutgers, Buitenzorg, Java.

American Peat Society—At the 8th annual meeting of the Society, held in Duluth, Minn., Aug. 20-22, the following papers were presented:

"General Drainage Work, or Reclamation of Swamp Areas," Dr. Joseph Hyde Pratt, State Geologist of North Carolina; "Latest Development in the Peat Gas Producer," F. B. Haanel, Department of Mines, Ottawa, Canada; "The Possibilities of Peat in the Beneficiation and Smelting of Iron Ores," Prof. Peter Christiansen, University of Minnesota; "Converting Peat into Fuel, and the Origin and Nature of Muskeg Peat," Prof. Chas. A. Davis, U. S. Bureau of Mines; "The Peat Industry in Ireland," Thomas Tomlinson, Dublin, Ireland; "Peat Briquetting," Dr. J. McWilliam, London, Canada; "Briquettes from Peat and Petroleum," Robert Blei, Sacramento, Calif.; "Practical Demonstration of Herbein's Patent by a Small Working Model of the Herbein Briquetting Machine," C. A. Squire, Cleveland, Ohio; How to Prepare a Peat Bog for Fuel Making," Gustave Carlson, Farnham, Ont., Canada; "Application of Machinery on Peat Deposits," L. A. Krupp, Findlay, Ohio; "Economic Aspect of Peat in America," John N. Hoff, New York; "Reclamation of Swamp Lands of Carolina and the South Atlantic States," Prof. J. D. Wood, University of Nebraska; "Comparative Costs of Drying Peat Fertilizer with Fuel Oil and Coal," John Wiedmer; the Secretary is Julius Bordollo, Kingsbridge, New York City.

A City Managers' Convention will be held in Springfield, Ohio, Dec. 2-4. For information regarding the convention, Charles F. Ashburner, City Manager, Springfield, Ohio, should be addressed.